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FEASIBILITY STUDY: PROPOSED WET PARK AT THE TUCSON
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ENGINEERS TUCSON AZ NOV 79 DACW09-78-C-0041

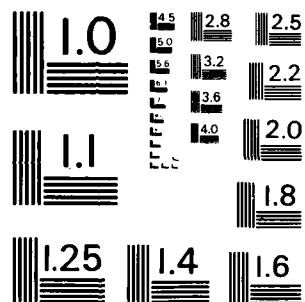
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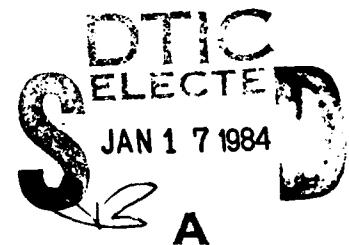
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AD A 136997

**FEASIBILITY STUDY:
PROPOSED WET PARK
AT THE TUCSON DETENTION
BASIN, PIMA COUNTY, ARIZONA**

**PREPARED FOR
U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT**

**PREPARED BY
RGA CONSULTING ENGINEERS
TUCSON, ARIZONA**



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FEASIBILITY STUDY - PROPOSED WET PARK

AT THE

TUCSON DETENTION BASIN, PIMA COUNTY, ARIZONA

Prepared For

DEPARTMENT OF THE ARMY

LOS ANGELES DISTRICT, CORPS OF ENGINEERS

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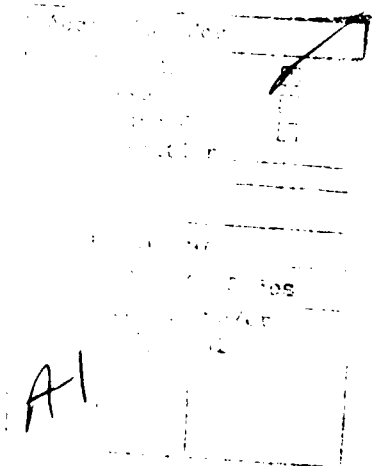
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EXECUTIVE SUMMARY

Discussion of Report

Part One: This report examines the feasibility of constructing a recreational lake and related dryland facilities at the Tucson Detention Basin Flood Control Project, using treated effluent from the Randolph Park Wastewater Treatment Plant to fill and maintain a recreational lake.

While the basin will continue to serve its primary function as a flood control device, the lake would serve a second, much needed, function: as a recreational facility, for boating, fishing, and limited continuous body contact activities.

One plan suggests that the lake initially be filled with groundwater, only later utilizing treated effluent from the Randolph Park Wastewater Treatment Plant (1.75 miles northeast of the detention basin) for water level maintenance.

Another plan recommends that the lake be filled and maintained with treated effluent not used for Randolph Golf Course; Randolph (Reid) Park would then continue, as it does currently, to use groundwater for its irrigation.

Part Two: The detention basin is located within the Santa Cruz River drainage area, in which the predominant soil materials are clayey sands, irregular occurrences of sandy clays, and borderline sands having a 3% to 19% moisture content.

Due to an absence of cacti (saguaro or other), a desert "indicator," and the presence of trees such as cottonwood and mesquite, this site is significantly different from the surrounding environment. It might be appropriate to call it a transitional zone, an area between desert-grasslands and forest zones, attributable to the additional moisture available in the detention basin.

The proposed recreational lake is not likely to destroy or seriously disrupt rare or endangered flora and fauna. Various commercial, industrial, and residential activities have, in the past, sufficiently disturbed this area so that it is now reasonable to assume that any damage to rare and endangered species has already been done.

Part Three: A comparison of the physical, biological, and chemical properties of the Randolph Wastewater Treatment Plant effluent with the water quality requirements for recreational use (as established by the Arizona Department of Health Services) reveals that a reduction in suspended solids and nutrients is necessary to meet the

recreational water quality requirements. (Treatment alternatives are discussed in Part 5.) Furthermore, since the Randolph Wastewater Treatment Plant serves an established residential area, the overall quality of the effluent is not anticipated to change.

Part Four: Due to an inadequate sewage collection system, the maximum influent that can be delivered to the Randolph Park Wastewater Treatment Plant falls short of the plant's design capacity by 0.5 mgd. Since no effort to correct this inadequacy is foreseen, all calculations used to determine the quantity of effluent available for the Ajo Wet Park have been based on the current influent and effluent rates.

Water losses in the recreational lake due to evaporation, vegetation transpiration, and so on, total approximately 10% of the flow from the Randolph plant. Based on the current Randolph Park Wastewater Treatment Plant operating level, and considering the water loss rate, the maximum lake size was calculated to be 8.0 acres. Using the more substantial Randolph plant capacity figure (1.5 mgd), the maximum lake size is then calculated to range from 61 to 98 acres depending upon the quantity of additional effluent that will be made available for the lake.

Stormwater and Tucson Electric Power Company blowdown water were also considered, but rejected, as possible sources for the Ajo Wet Park.

Part Five: There are various processes that may be used to bring the Randolph Park effluent up to the required water quality standards. Land treatment, filtration, microscreening, carbon adsorption, phosphorus removal, nitrogen removal, and the use of polishing ponds are tertiary processes worthy of consideration. However, due to the high initial cost, carbon adsorption might prove an unattractive alternative.

Likewise, microscreen units are expensive and require additional operating costs. Polishing ponds, on the other hand, can reduce the settleable solids, BOD, and fecal coliforms, and, with the introduction of fish, could serve as a nonboating fishing area.

Part Six: The quantity and quality of effluent processed at the Randolph Wastewater Plant are inadequate to maintain a fishing and boating lake of the Ajo Detention Basin without extensive capital improvements. These capital improvements include a 0.5 mgd raw sewage pumping station; a 0.81 mgd two stage tertiary lime treatment plant to reduce phosphorous and heavy metals; an ammonia stripping plant to remove ammonia and reduce total nitrogen; and a filtration plant to reduce turbidity and suspended solids.

PART 1

GENERAL

A. PURPOSE:

The purpose of this report is to study the feasibility of using treated effluent from the Randolph Park Wastewater Treatment Plant to fill and maintain a recreational lake and park located at the Tucson Detention Basin Flood Control Project. The lake should be adequate for fishing and boating with only incidental body contact. The results of this study will be used to make future planning and design decisions concerning the Ajo Wet Park project.

B. SCOPE:

The objectives of the report are as follows:

1. Study and discuss the properties of the proposed Ajo Wet Park site.
2. Study and discuss the quantity and quality of the Randolph Park Wastewater Treatment Plant Effluent.
3. Estimate the existing and future quantity of effluent available for use in the Ajo Wet Park Lake.
4. Prepare alternate concepts, with estimate of the construction, operating and maintenance costs for treatment of secondary plant effluent for use at the Ajo Wet Park Lake.
5. Discuss the feasibility of lake development.

C. INTRODUCTION:

The Pima County Parks and Recreation Department, in cooperation with the U. S. Army Corps of Engineers and the City of Tucson, has developed plans for a recreational lake and related dryland park facilities to be built at the Tucson diversion channel flood control basin.

The project is located in eastern Pima County, just outside the city limit between Campbell Avenue and Country Club Road (Figure 1.1). While simultaneously maintaining the basin's primary function as a flood control device, the lake is to be established to provide recreational boating and fishing for the

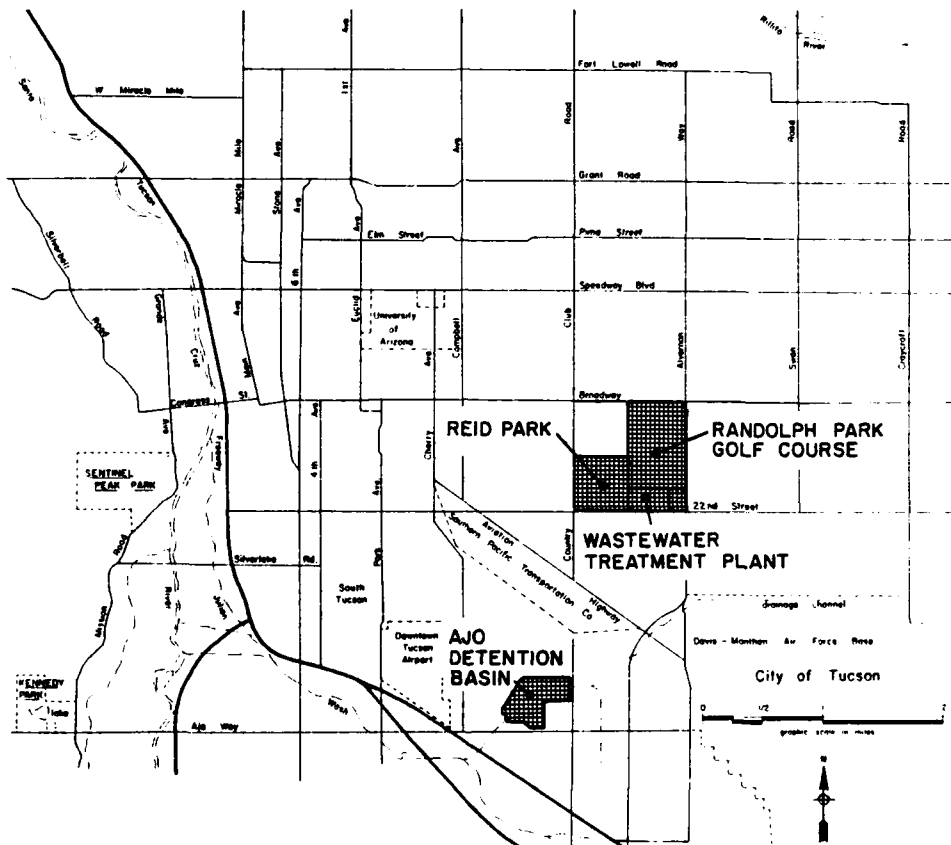


FIGURE 1.1
PROJECT LOCATION

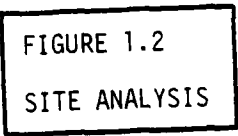
immediate area, which has been shown, by surveys and public meetings, to be significantly lacking in such facilities. Swimming and other continuous body contact with the lake water, however, will not be encouraged.

The lake could initially be filled with water pumped from the ground, but once full the water level would be maintained with treated effluent from the Randolph (Reid) Park Wastewater Treatment Plant, which is located about 1.75 miles northeast of the detention basin at the intersection of 22nd Street and Alvernon Way.

The existing site has been analyzed by the Pima County Parks and Recreation Department and the Corps of Engineers; a rough illustration of this analysis is in Figure 1.2. Figure 1.3 represents the topography of the existing detention basin and its associated levees and drainage channels.

The proposed park will be planted with native vegetation and will feature a boat launching area, fish cleaning station, trails for hiking and bicycling, restrooms, ramadas, parking lot and a sewage treatment area. A conceptual drawing of the completed park with a 60 acre lake is in Figure 1.4.

The original design developed by the Corps of Engineers and Pima County Parks and Recreation Department included a pump station and pipeline to return water from the lake to Randolph (Reid) Park for irrigation. An alternate plan suggests that effluent not used for Randolph golf course be used to fill the lake; Randolph (Reid) Park would continue, as it does currently, to use groundwater directly for its irrigation.



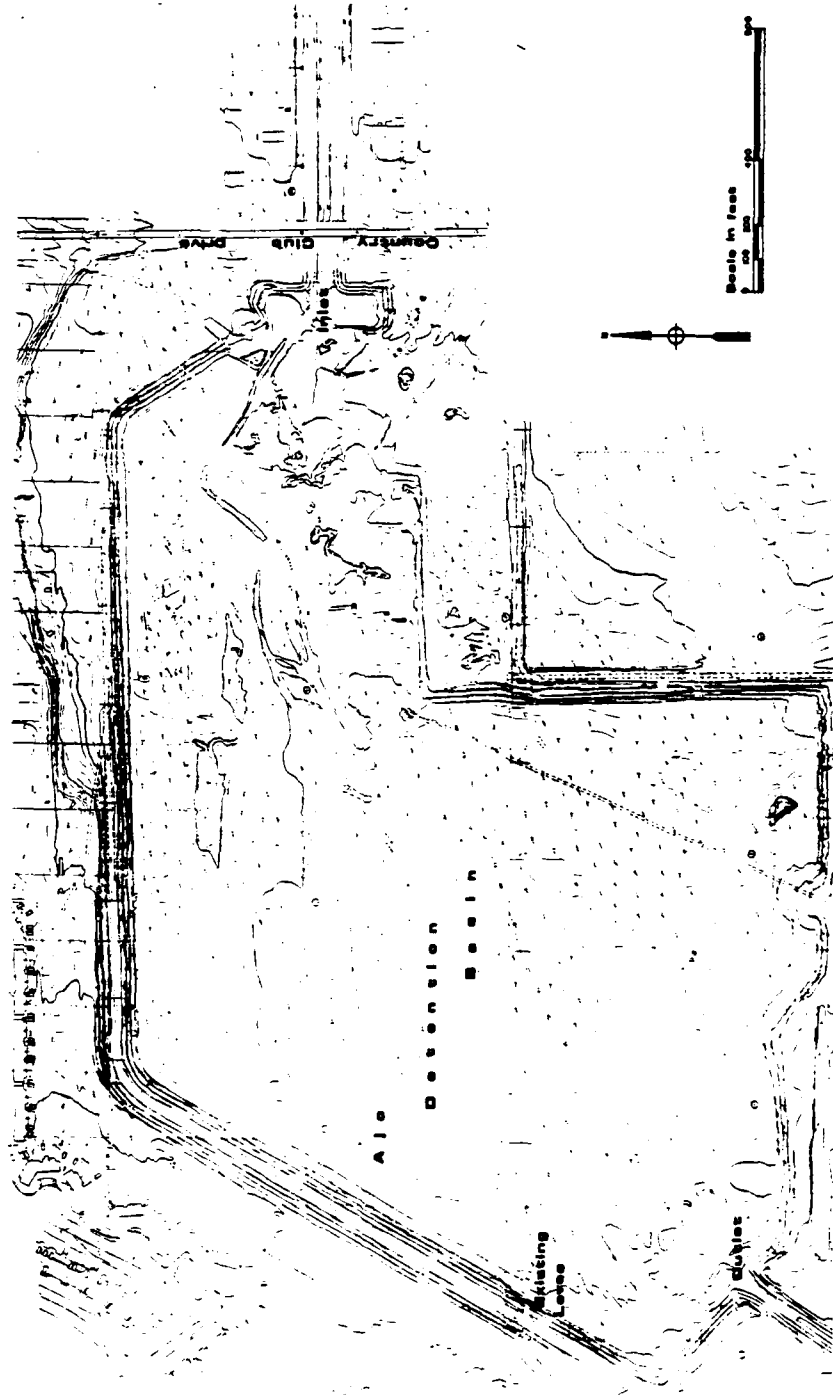


FIGURE 1.3
TOPOGRAPHY

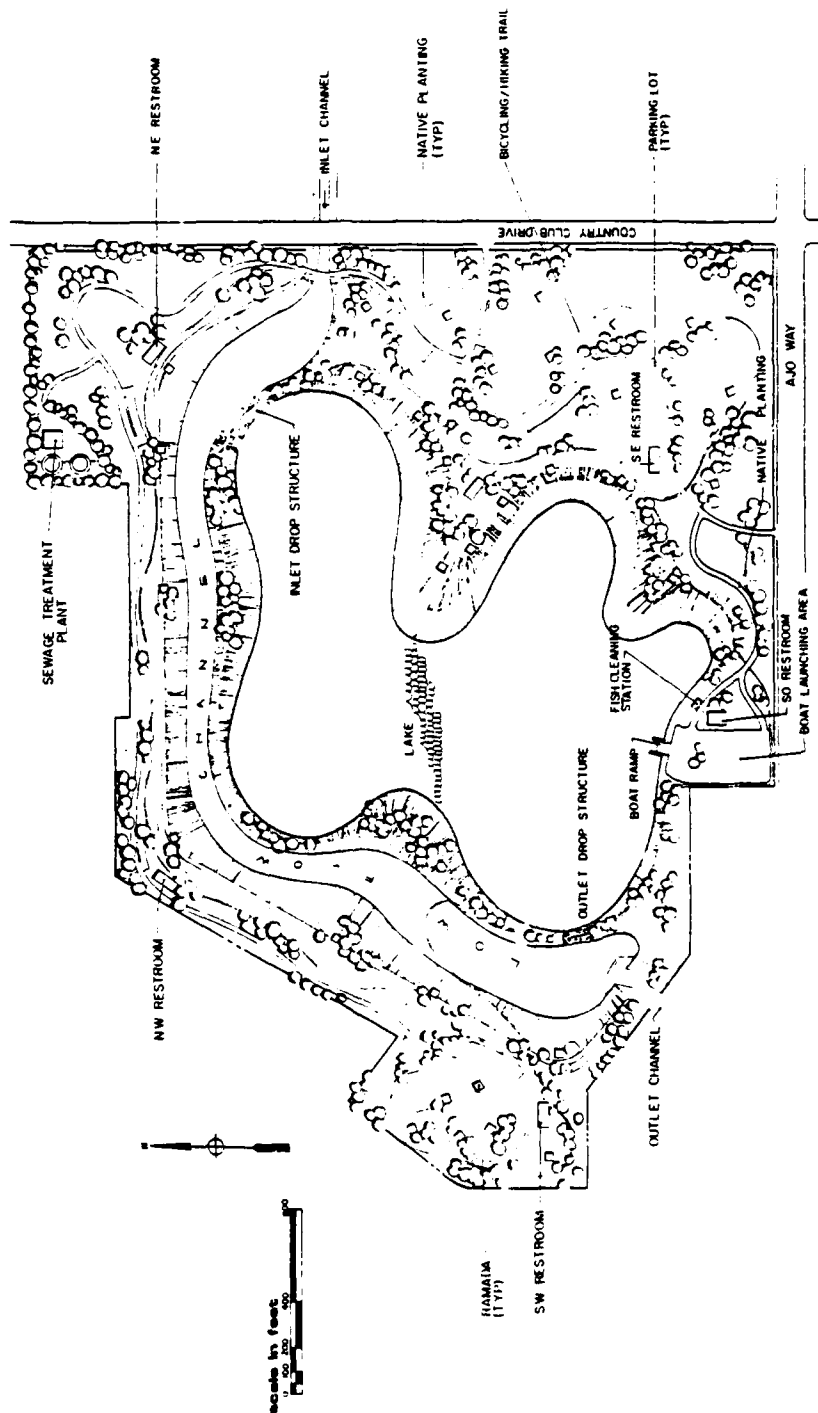


FIGURE 1.4
PROPOSED PARK WITH
60 ACRE LAKE

PART 2
PRESENT PHYSICAL, CHEMICAL
AND
BIOLOGICAL PROPERTIES OF THE DETENTION BASIN SITE

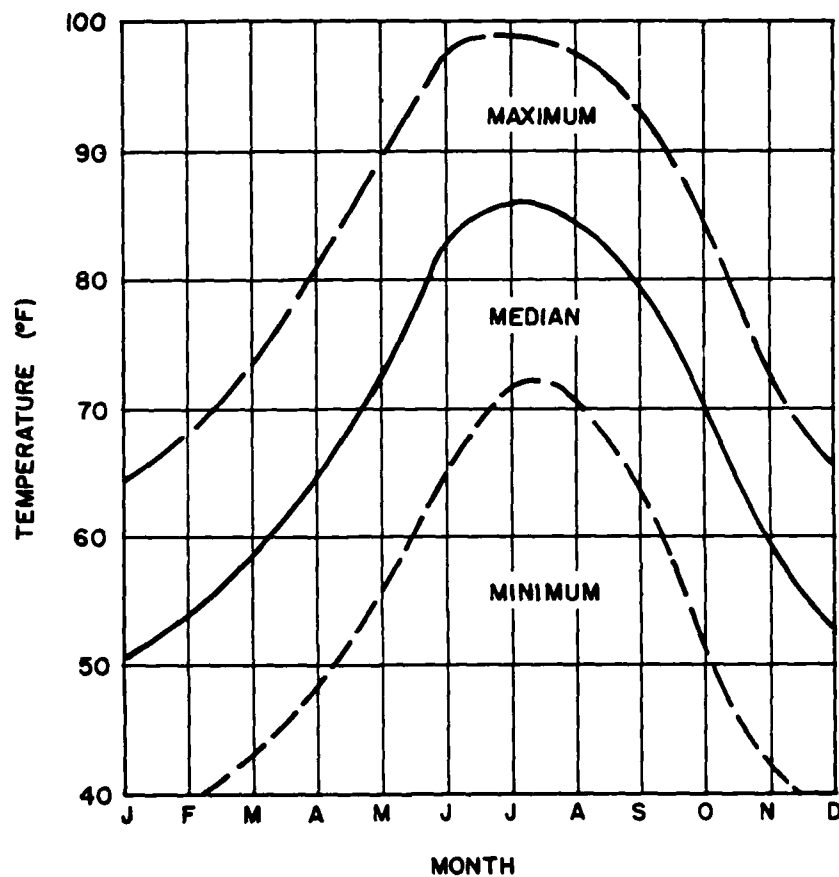
The detention basin is approximately 120 acres in area with a perimeter consisting of a 20 ft. wide berm. The low flow channel through the basin is about 2447 ft. long from inlet to outlet. The inlet structure, on the northeast side of the basin, is 46 feet across the channel bottom at its opening into the basin, and the outlet structure, on the southwest side, is 60 ft. across, narrowing to 15 ft. entering the outlet channel. A 24" corrugated metal pipe overflow structure is imbedded in the berm (levee) on the southwest perimeter, about half-way between the inlet and outlet structures.

There is a sand extraction operation and borrow pit excavation in the northeast part of the basin, about 500 ft. from the inlet, and a chip stockpile southwest of the inlet used by the Pima County Highway Dept. In addition, concrete and asphalt debris has been dumped in this same general area. None of these operations will deter or affect the construction of a lake and park.

The basin is situated within the Santa Cruz River drainage area. Out of 10 soil test holes drilled to a depth of 30 ft., none encountered groundwater; however, a 1972 U.S.G.S. map of depth to water in wells indicates a probable depth to water of 100 to 200 ft. in the basin area. The predominant materials in the test hole samples were clayey sands with irregular occurrences of sandy clays and borderline sands having a 3% to 19% moisture content range, with a moisture content of 10% being average. Materials in the southern half of the basin are cemented to some degree by caliche, and caliche (a soil cemented by calcium carbonate) forms a solid pavement in front of the inlet (Soils Investigation Report, Corps of Engineers).

Surface winds in this region of Arizona are light most of the time, with an average annual velocity of about 8.1 mph from the southeast. There are no important seasonal changes in either prevailing direction or velocity, highest velocities usually occurring with winds from the southwest and east to south. Maximum velocities at Tucson International Airport reach 59 mph, with an annual average maximum velocity of about 35 mph (NOAA Climatological Data). Temperature data is in Figure 2.1 and information on sunshine is in Table 2.1.

There are two Hohokam sherd sites, ca 900-1300 A.D., located within about one mile of the detention basin. One of the sites, currently occupied by an abandoned service station, is at the southwest corner of Ajo Way and Palo Verde Road; the other is located north of Irvington Road, between Country Club Road and I-10, on the north bank of Julian Wash.



**AIR TEMPERATURES FOR THE TUCSON
METROPOLITAN AREA**

SOURCE: UNITED STATES WEATHER BUREAU RECORDS

FIGURE 2.1

**AIR TEMPERATURES FOR THE
TUCSON METROPOLITAN AREA**

PERCENT OF POSSIBLE SUNSHINE AT TUCSON INTERNATIONAL AIRPORT													
MONTH	J	F	M	A	M	J	J	A	S	O	N	D	ANNUAL AVERAGE
% OF POSSIBLE SUNSHINE (1948-1954 AVERAGE)	78.6	84.0	82.8	88.0	93.0	93.3	73.8	82.8	82.4	91.3	90.1	80.0	85.0

Source: The Climate Of Arizona, H. V. Smith, 1956.

TABLE 2.1
PERCENT OF POSSIBLE
SUNSHINE AT TUCSON
INTERNATIONAL AIRPORT

There are, however, no records of archaeological sites or other cultural resources within the detention basin itself.

Furthermore, there are no mineral deposits within the basin site. The soil is strongly calcareous, and is moderately alkaline with a pH between 7.9 and 8.4. It has a high corrosivity to uncoated steel and a low corrosivity to concrete. Since the lake will be lined, these properties should not affect the quality of the lake water.

Although disturbed in recent years, this site appears to be a desert-grassland biotic community of the high Lower Sonoran and low Upper Sonoran life zones. The former life zone is represented by plants such as paloverde (Cercidium), creosotebush (Larrea), and mesquite (Prosopis), while shrubs (e.g. Baccharis, Psilostrophe) and grasses (e.g. Sorghum, Phalaris) comprise the latter zone. There are two main reasons for describing this community as a combination of desert and grassland. One is the lack of saguaro cacti, a Sonoran desert "indicator" species. In fact, no cacti of any species are found in the detention basin. Secondly, at altitudes below 3,000 ft., grassland shrubs and grasses flourish in the low, flat areas frequently inundated by seasonal storms.

The absence of cacti and the presence of large trees (i.e. Populus fremontii) usually found near streams make this area (elevation 2500') significantly different from the surrounding environment. It might be appropriate to call it a transitional zone, an area between desert-grassland and forest zones, attributable, clearly enough, to the additional moisture available in the detention basin. Moreover, the deciduous riparian forest community includes cottonwood (Populus), mesquite, and blue paloverde, which also grow in artificially created habitats where human activities have changed the drainage pattern of the landscape (Turner, 1974).

The existing basin and levees have obviously affected the distribution and composition of local vegetation. Although most of the levees are sparsely covered with grasses and shrubs, the inner slope of the northwest levee has a noticeably denser cover of creosotebush, paloverde, and mesquite trees, due to the prevailing pattern of drainage toward the Santa Cruz River. Also, plants (e.g. Baccharis, or Desertbroom) in the bottom of the basin are gradually being buried by sediments brought in when the basin is flooded by storm runoff.

During the period of initial observations, jack rabbits (Lepus), quail (Lophortyx gambelii), and a few lizards were seen. It is reasonable to assume that animals such as javelina (Pecari) and deer (Odocoileus) once inhabited the site but have since been displaced by urbanization. In addition, this nearly riparian forest habitat, though artificial, is probably a nesting site for game birds including whitewing doves (Zenaidura asiatica), and the previously mentioned Gambel quail.

The proposed recreational lake construction is not likely to destroy rare or endangered fauna or flora. Previous ecological studies have shown that the Arizona desert flora has relatively little true endemism (Lowe, 1964). The detention basin site has no features which qualify it as a unique habitat. There are no geographical or climatic isolating mechanisms that could allow speciation to occur through genetic divergence between populations of a given organism. Since the construction of the lake and its supporting facilities will not occur near the basin inlet, the riparian forest habitat should remain undisturbed. Furthermore, the area has already been severely disturbed by various commercial, industrial, and residential activities; it is reasonable to assume, therefore, that the damage to rare and endangered species has already been done. In addition, disturbances of either natural or artificial habitats often lead to crossbreeding (Blair, 1961).

Grasses likely to colonize the lake shore are those found at the basin bottom adjacent to the inlet (e.g. Sorghum halapense), and in other low spots where water tends to accumulate (e.g. Phalaris caroliniana). Some of the Sorghum was infected with a fungus resembling ergot (which is any fungus of the genus Claviceps), many of the seeds being replaced by black, powdery masses which are probably the dried sclerotia of the fungus.

Finally, it is important to note that this report is based on observations made during the late spring flowering season. Vegetation is more abundant at this time of year than at any other. On the other hand, while some plants bloom at any time of the year and may appear to be more abundant than this report would indicate, others bloom earlier or later and are not included at all.

PART 3

QUALITY OF RANDOLPH EFFLUENT

The physical, biological, and chemical properties of the effluent from the Randolph Wastewater Treatment Plant are shown in Table 3.1. The type of effluent treatment that is needed for these properties are compared to the water quality requirements for recreational use established by the Arizona Department of Health Services. These water quality standards are shown in Tables 3.2 and 3.3. These wastewater reuse guidelines were based on (1) the "Rules and Regulations for Reclaimed Wastes," Article 6, Part 4, by the Arizona State Department of Health, in which the minimum level of treatment specified is "secondary treatment," and (2) specific Federal criteria for what constitutes "secondary treatment". It should be mentioned that the EPA guidelines are more stringent than those of the State, 30 mg/l BOD compared to 35 mg/l BOD, and 30 mg/l total SS compared to 35 mg/l total SS for the state. These two values have been incorporated in Table 3.2, the values in parentheses reflect the anticipated change in the State guidelines to match the existing, more rigorous Federal guidelines.

An analysis of these three tables reveals that by reducing the level of suspended solids and nutrients, the effluent would meet recreational water quality requirements. The treatment process that could be used are discussed in Chapter 5.

The residential area that the Randolph plant serves is well established and not likely to change radically in the near future. Therefore, no major changes in the quality of the effluent during the life of the wastewater treatment plant are anticipated.

The following is a description of some of the parameters given in Table 3.1 which are used to evaluate the suitability of effluent for reuse:

- A. Dissolved solids is a general term used to describe the mineral content of water. Total dissolved solids (TDS) consist primarily of sodium, potassium, calcium and magnesium cations and carbonate, chloride, sulfate and nitrate anions. Other constituents usually present in small amounts may be silver, arsenic, iron, chromium, cadmium, lead, mercury, copper, zinc, etc. Generally speaking, water with a TDS of less than 1000 mg/l is considered fresh; a TDS from 1000 to 10,000 mg/l is considered brackish; a TDS from 10,000 to 25,000 mg/l is considered saline; and a TDS greater than 25,000 mg/l is considered seawater.
- B. Biological Oxygen Demand (BOD₅) is the most widely used parameter in describing organic pollution, applied to both wastewater and

QUALITY OF EFFLUENT

PARAMETER	RANDOLPH EFFLUENT (Average)		
Fecal Coliform N/100 m.l.	279		
5 Day BOD mg/l	15		
Dissolved Oxygen	1.8		
Turbidity Jackson Turbidity Units	30		
pH	7.4		
Suspended Solids mg/l	14		
Settleable Solids mg/l	6.0		
Chlorine Residual mg/l	0.9		
Total Dissolved Solids mg/l	486		
Phosphates (as PO_4) mg/l	18.2		
Iron (mg/l)	0.65		
Nickel (mg/l)	0.06		
Cadmium (mg/l)	0.008		
Chromium (mg/l)	0.03		
Copper (mg/l)	0.36		
Zinc (mg/l)	0.39		
Lead (mg/l)	0.04		
Manganese (mg/l)	0.05		
NITROGEN SPECIES	HIGH	LOW	MEDIAN
Ammonium Nitrogen (as N) mg/l	45.3	7.1	20.2
Nitrate Nitrogen (as N) mg/l	2.6	.14	1.1
Nitrite Nitrogen (as N) mg/l	5.8	.02	1.5

TABLE 3.1

QUALITY OF EFFLUENT

EFFLUENT QUALITY REQUIREMENTS FOR VARIOUS USES ^a							
USE	BOD ₅	Total SS	Total dissolved solids	Toxic substance	Total Phosphorus	Total nitrogen	Bacteriological
<u>IRRIGATION</u>							
Fibrous or forage crops not intended for human consumption	35 (30)	35 (30)	709	a	b	b	b 1000
Orchard crops-no direct application of water to fruit or foliage	35 (30)	35 (30)	709	a	b	b	b 1000
Food crops-product subjected to physical or chemical processing sufficient to destroy pathogenic organisms	35 (30)	35 (30)	709	a	b	b	b 1000
Orchard crops-direct application to fruit or foliage	35 (30)	35 (30)	709	a	b	b	b 1000
Food crops which may be consumed in their raw state	10	10	709	a	b	b	200
Golf courses, cemeteries and similar areas	35 (30)	35 (30)	709	a	b	b	b 1000
School grounds, playgrounds, etc. where children are expected to play	10	10	709	a	b	b	200
<u>WATERING</u>							
Farm animals other than producing dairy animals	35 (30)	35 (30)	709	a	b	b	b 1000
Producing dairy animals	35 (30)	35 (30)	709	a	b	b	1000
<u>RECREATIONAL IMPOUNDMENTS</u>							
Aesthetic enjoyment or involving only secondary contact	35 (10)	35 (10)	709	a	b (.15)		1000 (200)
Primary contact recreation	10 (5)	10 (5)	709	a	0.5	b	(2.0)
<u>GROUNDWATER RECHARGE</u>							
Ponding on surface	35 (30)	35 (30)	409	a	b	b	1000
Well-point	10 (5)	10 (5)	409	a	0.5	10	200

^a-Concentrations expressed in terms of mg/l.

-Bacteriological figures expressed in terms of fecal coliform group density (count) per 100 milliliters.

a) Not to exceed United States Health Service drinking water standards.

b) No limit on concentration.

c) Based on "Effluent Parameters for Reclaimed Wastes," by Arizona Department of Health, April, 1972.

*) Obtained by telephone.

Figures in parentheses are anticipated future standards.

TABLE 3.2

EFFLUENT QUALITY REQUIREMENTS FOR VARIOUS USES

PARAMETER	SPECIFIC STANDARDS FOR PROTECTED USES					
	DOMESTIC WATER SOURCE	RECREATION		PROTECTED USES		LIVESTOCK WATERING
		FULL BODY	PARTIAL BODY	AQUATIC AND WILDLIFE	IRRIGATION	
FECAL COLIFORM ^a (UNITS / 100 ml)						
1. GEOMETRIC MEAN (5 SAMPLE MINIMUM)	1000	200	1000	1000	1000	1000
2. 10% OF SAMPLES FOR 30 DAY PERIOD SHALL NOT EXCEED	2000	400	2000	2000	2000	2000
3. SINGLE SAMPLE SHALL NOT EXCEED	4000	800	4000	4000	4000	4000
pH ^f						
1. MAXIMUM	NS	8.6	8.6	9.0	9.0	9.0
2. MINIMUM	NS	6.5	6.5	6.5	4.5	6.5
3. MAXIMUM CHANGE DUE TO WASTE DISCHARGE	NS	0.5	0.5	0.5	NS	NS
TRACE SUBSTANCES ^f (MAXIMUM MG/L)						
ARSENIC (AS AS)	0.050 D	0.050 D	b	0.050 D	2.000 T	0.200 T
BARIUM (AS Ba)	1.000 D	1.000 D	---	NS	NS	NS
BORON (AS B)	NS	NS	b	NS	1.000 T	NS
CADMIUM (AS Cd)	0.010 T	0.010 T	---	0.010 D ^c	0.050 T	0.050 T
CHROMIUM (AS Cr, HEXAVALENT & TRIVALENT)	0.050 D	0.050 D	---	0.050 D	1.000 T	1.000 T
COPPER (AS Cu)	NS	NS	---	0.050 D	5.000 T	0.500 T
LEAD (AS Pb)	0.050 D	0.050 D	---	LESS THAN 0.050 D ^e	10.000 T	0.100 T
MANGANESE (AS Mn)	NS	NS	---	NS	10.000 T	NS
MERCURY (AS Hg)	0.002 T	0.002 T	---	LESS THAN 0.002 T ^e	---	0.010 T
SELENIUM (AS Se)	0.010 D	0.010 D	---	---	0.020 T	0.050 T
SILVER (AS Ag)	0.050 D	0.050 D	---	0.050 D	NS	NS
ZINC (AS Zn)	NS	NS	---	0.500 D	10.000 T	NS
AMMONIA (AS UN-IONIZED NH ₃)	NS	NS	NS	0.020	NS	NS
CYANIDES (AS CYANIDE ION & COMPLEXES)	0.200	0.200	NS ^b	LESS THAN 0.020 ^e	NS	0.200
PHENOLICS	0.005	0.005	---	0.005	NS	0.005
SULFIDES (TOTAL)	NS	NS	NS	LESS THAN 0.100 ^e	NS	NS

NOTES

- For limits applicable to direct wastewater reuse, see A.C.R.R. R9-20-400's.
- Too little is known about adverse effects for this use to adequately select a number.
- For cold water fishery habitat, maximum cadmium concentration is 0.001 mg/l.
- Abbreviation used in this table: NS - NO STANDARD, T - TOTAL TRACE SUBSTANCES, D - DISSOLVED FRACTION
- The maximum concentration for this use is set at the current minimum level of detection. The maximum concentration necessary to adequately protect this use is lower.
- Applies also to Effluent Dominated Streams.

SOURCE: Water Quality Standards for Surface Waters, Arizona Water Quality Control Council, June 8, 1979.

TABLE 3.3a
SPECIFIC STANDARDS
FOR PROTECTED USES

SPECIFIC STANDARDS FOR PROTECTED USES - contd

PARAMETER	DOMESTIC WATER SOURCE	PROTECTED USES				AGRICULTURAL IRRIGATION AND LIVESTOCK WATERING
		RECREATION FULL AND PARTIAL BODY	WARM WATER FISHERY HABITAT	AQUATIC AND WILDLIFE COLD WATER FISHERY HABITAT		
TEMPERATURE ^{f, h} HEAT ADDED BY A DISCHARGE OR COMBINATION OF DISCHARGES SHALL NOT RAISE THE NATURAL AMBIENT WATER TEMPERATURE MORE THAN DEGREES CELSIUS	NS	3	3	1	NS	
TURBIDITY ^{f, i} A DISCHARGE OR COMBINATION OF DISCHARGES SHALL NOT CAUSE THE TURBIDITY TO EXCEED JACKSON TURBIDITY UNITS IN: STREAMS - LAKES -	NS NS	50 25	50 25	10 10	NS NS	
DISSOLVED OXYGEN ^g A DISCHARGE OR COMBINATION OF DISCHARGES SHALL NOT LOWER THE DISSOLVED OXYGEN CONCENTRATION TO LESS THAN MG/L	NS	6	6	6	NS	

NOTES

- f. Applies also to Effluent Dominated Streams.
- g. Does not apply to Effluent Dominated Streams.
- h. Temperature standard not applicable to impoundments owned by a firm or individual for the express purpose of providing or receiving heat wastes.
- i. Standards are applicable to turbidity caused by activities including, but not limited to, construction, mining, logging, agriculture, and other similar non-point sources.
- j. Abbreviations used in this table: NS - NO STANDARD

SOURCE: Water Quality Standards for Surface Waters, Arizona Water Quality Control Council, June 8, 1979.

TABLE 3.3b

SPECIFIC STANDARDS
FOR PROTECTED USES

surface waters. This parameter is a determination of the relative amount of dissolved oxygen that is used by micro-organisms in the biochemical oxidation of organic matter.

- C. Suspended solids (SS) generally describes the organic and inorganic particles that are not dissolved. Approximately 75% of suspended solids are organic in nature, generated by both plant and animal life. Organic compounds consist of combinations of carbon, hydrogen and oxygen. Other elements such as sulfur, phosphorus and iron may also be present. Suspended solids also encompass an ever increasing amount of synthetically produced organics ranging from very simple to extremely complex in structure. These synthetically produced organics include substances used as surfactants, phenols and agricultural pesticides. The presence of these substances has complicated wastewater treatment in recent years because many of them cannot be, or are very slowly, decomposed biologically.
- D. The Fecal Coliform count is a measurement that generally indicates micro-biological content including viruses and pathogenic organisms. Fecal Bacteria of the coliform group are primary indicators of fecal contamination and are of sanitary significance. Fecal coliform bacteria is often used to monitor recreational water quality.
- E. Phosphorus in its elemental form can be toxic to man and accumulates in much of the same way as mercury. Phosphorus as phosphate is a nutrient which is essential for plant life. Phosphate stimulates growth of aquatic plants such as algae which can result in eutrophication.
- F. Nitrogen comes in several forms - two gases, molecular nitrogen and nitrous oxide, and also in five nongaseous forms of combined nitrogen, ammonia nitrite and nitrate, and amino and amide groups all of which are a significant part of the nitrogen cycle.

Ammonia, organic nitrogen, nitrates and nitrites are the forms of nitrogen that are significantly present in wastewater. Organic nitrogen and ammonia both of which are discharged in human wastes are, generally speaking, the initial forms of nitrogen present in sewage. As time progresses, bacterial action converts the organic nitrogen into ammonia, and then, under aerobic conditions, the ammonia is oxidized to nitrites and nitrates. Under anaerobic conditions nitrates are reduced to nitrites. Nitrites under anaerobic conditions will be further reduced to nitrogen gas, or to a lesser degree, to ammonia. The relative proportions of these forms of nitrogen, therefore, are indicators of the freshness of wastewater and the quality of treated effluent.

Organic, ammonia and nitrite nitrogen present in wastewater treatment plant effluent will exert an oxygen demand in the receiving waters. In addition, the nitrate form of nitrogen will serve as a nutrient for aquatic plants and will promote eutrophication of lakes.

PART 4
QUANTITY OF EFFLUENT

RANDOLPH PARK EFFLUENT:

The proposed source of water for the Ajo Wet Park is the Randolph wastewater treatment plant. This facility is an activated sludge plant designed to treat 1.5 mgd. Presently, the maximum influent that can be delivered to the plant is approximately 1.0 mgd. This is due to the inadequacy of the existing sewage collection system.

Mr. Dave Johnson, of the Tucson Water and Sewer Department, stated that by constructing a lift station and one-half mile of pipeline, additional flow could be obtained. He estimated that there would be enough additional influent to run the Randolph plant at its maximum capacity. However, since it is unknown whether or not the funding for the lift station and pipeline can be obtained, calculations for the size of the lake will cover the following three possible situations:

1. Existing conditions remain the same.
2. Flow to the Randolph Wastewater Treatment Plant is increased and the effluent in excess of the golf course requirements is utilized by the proposed lake.
3. Flow to the Randolph Wastewater Treatment Plant is increased and all of the additional effluent is utilized by the proposed lake.

The size of the proposed lake is determined by evaporation, seepage, precipitation, and the available quantity of effluent.

EVAPORATION:

The loss of water due to evaporation was determined using pan evaporation information from the National Ocean and Atmospheric Administration (NOAA) Environmental Data Service. This data was collected at the University of Arizona weather station. These values were then multiplied by 0.7 to obtain approximate lake evaporation rates (0.7 is the conversion factor recommended by the University of Arizona to convert from pan evaporation rates to lake evaporation rates). Both the pan and lake evaporation figures are shown in Table 4.1.

SEEPAGE:

Due to the permeability of the sands which make up the floor of the basin, it is assumed that the lake bottom would be sealed. If the lake bottom were sealed with a silty clay, the expected loss would be a drop in water elevation of approximately one foot per year.

PAN EVAPORATION (INCHES/MONTH)													
YEAR	J	F	M	A	M	J	J	A	S	O	N	D	ANNUAL
1978	3.29	4.16	7.58	10.87	13.84	17.36	16.18	14.18	13.33	9.76	4.36	3.36	118.27
1977	3.09	5.62	8.54	10.77	13.73	17.09	14.63	13.17	11.13	8.44	5.46	4.26	115.93
1976	4.53	6.20	9.23	11.37	14.87	17.83	13.05	14.63	10.48	8.40	6.10	4.42	121.11
1975	4.46	5.03	8.02	9.53	13.79	17.27	14.28	14.53	12.38	10.35	7.10	3.60	120.34
1974	3.95	6.16	8.55	11.90	16.61	20.24	15.30	13.10	10.86	8.40	4.19	3.41	122.67

SOURCE: Data Collected At The University of Arizona Weather Station

AVERAGE EVAPORATION PER MONTH FOR THE PERIOD 1974-1978 (INCHES)													
MONTH	J	F	M	A	M	J	J	A	S	O	N	D	ANNUAL
Pan Evaporation	3.86	5.43	8.38	10.88	14.56	18.07	14.68	13.92	11.63	9.07	5.44	3.81	119.66
Lake Evaporation	2.70	3.80	5.87	7.62	10.1	12.6	10.3	9.74	8.14	6.34	3.80	2.66	83.8

SOURCE: Climatological Data, NOAA Environment Data Service

TABLE 4.1
EVAPORATION DATA

OTHER LOSSES:

Other losses that might occur from the proposed project include vegetation transpiration, tertiary treatment process losses in the transmission of the effluent from Randolph to the Ajo Wet Park, and evaporation at the Randolph plant. These losses were added together and approximated to be 10% of the flow from Randolph plant on the following basis:

a. Filter Backwash - 5% loss.

b. Leakage - At a pressure of 80 psi the allowable loss in an 8" diameter pipe is 1.08×10^{-3} gal/ft/hour (AWWA). Therefore, the annual loss would be:

$$\frac{(1.08 \times 10^{-3} \text{ gal/ft/hr}) (24 \text{ hrs}) (365 \text{ days}) (5000 \text{ ft})}{(7.48 \text{ gal/ft}^3) (43,560 \text{ ft}^3/\text{acre})}$$

$$= 0.15 \text{ acre/feet/year loss}$$

c. Vegetation Transpiration - Assume vegetation occurs over 15% of lake area. The consumptive use of this vegetation will be a drop in water elevation of 2.5 feet per year. (Irrigation Principles and Practices). Therefore, the annual loss would be:

$$(15\%) (2.5 \text{ feet}) (\text{Lake Size}).$$

$$\text{Since the } (\text{Lake Size}) = \frac{(\text{Effluent Volume})}{(\text{Lake Depth})}, \text{ and the } (\text{Lake Depth})$$

$$= 9 \text{ feet, the annual loss would be:}$$

$$\frac{(15\% (2.5 \text{ feet}))}{(9 \text{ feet})} (\text{Effluent Volume}) = 4\% (\text{Effluent Volume})$$

$$= 4\% \text{ loss}$$

PRECIPITATION:

The monthly precipitation data as obtained from the NOAA is shown in Table 4.2. The precipitation was measured at four locations around Tucson and averaged over five years from 1974 to 1978.

LAKE SIZE:

The calculation of lake size was done by computer, utilizing the computer program found in appendix B. This program takes the monthly effluent quantity, losses and minimum lake depth as input to calculate the surface area, volume and depth for each month. The lake is assumed to be twice as long as it is wide with 3 to 1 side slopes.

PRECIPITATION (INCHES/MONTH)													
STATION	YEAR	J	F	M	A	M	J	J	A	S	O	N	ANNUAL
Lampbell Avenue Experimental Farm EL. 2300	1978	2.76	2.35	2.47	0.23	0.89	0.41	1.62	2.21	0.14	2.13	2.04	20.99
	1977	2.09	0.05	0.64	0.30	0.10	0.06	0.94	0.25	0.70	3.56	0.59	10.72
	1976	0.16	0.36	0.52	1.23	0.33	0.07	0.97	0.26	1.98	2.06	0.47	7.76
	1975	0.46	0.13	0.79	0.92	0.00	0.00	1.96	0.53	1.01	0.00	0.26	6.81
	1974	0.71	0.00	0.62	0.00	0.00	0.00	1.38	2.34	3.98	2.57	0.66	12.57
Lucson Magnetic & Seismological Observatory EL. 2526	1978	3.11	2.69	2.28	0.13	0.77	0.25	1.54	2.40	0.83	2.15	2.67	22.53
	1977	3.22	0.00	0.76	0.24	0.05	0.25	1.66	1.11	1.05	2.37	0.34	12.66
	1976	0.24	1.02	0.50	0.94	0.36	*0.00	1.56	0.54	4.40	0.66	0.48	10.77
	1975	0.51	0.18	1.77	0.59	*0.00	0.00	3.01	0.32	1.38	0.27	0.50	9.05
	1974	1.30	0.02	0.88	*0.00	0.00	0.13	2.18	1.02	1.17	2.32	0.95	10.43
University Of Arizona EL. 2444	1978	2.69	2.44	1.46	0.30	0.84	0.79	1.40	1.66	0.10	2.59	1.78	19.54
	1977	1.91	0.03	0.68	0.34	0.11	0.59	1.10	0.78	0.89	2.71	0.54	11.02
	1976	0.35	0.32	0.60	1.14	0.47	0.35	2.01	0.52	2.29	0.51	0.37	9.55
	1975	0.61	0.06	0.95	0.60	*0.00	0.00	3.14	0.61	0.78	0.03	0.32	7.89
	1974	0.93	*0.00	0.38	*0.00	0.00	0.01	1.69	2.09	3.02	2.08	0.69	11.15
Weather Service Office Lucson Interna- tional Airport EL. 2585	1978	2.05	1.75	0.89	0.01	0.61	0.22	0.78	1.59	1.66	1.86	1.58	15.73
	1977	1.83	0.04	0.74	0.43	0.08	0.06	0.76	0.80	1.41	2.36	0.33	10.17
	1976	0.06	0.53	0.38	0.57	0.23	0.10	1.18	0.23	1.68	0.37	0.48	6.28
	1975	0.36	0.13	0.95	0.27	0.11	0.00	2.38	0.32	1.26	*0.00	0.34	6.64
	1974	0.93	*0.00	0.55	*0.00	0.00	0.01	4.44	1.04	1.69	2.12	0.81	11.92
Average Of All 4 Stations (Not Included In NOAA Data)		1.31	0.60	0.94	0.41	0.24	0.16	1.78	1.03	1.57	1.63	0.81	11.70

0.00 Underlined values are estimates (not included in NOAA data)

* Trace precipitation (less than 0.01 inch)

Source: Climatological Data, National Oceanic and Atmospheric Administration
Environment Data Service.

TABLE 4.2
PRECIPITATION

The most important factor that determines the size of the lake is the quantity of effluent. As mentioned previously, three possible situations which affect the quantity of effluent available for use at the proposed lake have been considered. The first situation (existing conditions remain unchanged) would result in the monthly water usages shown on Table 4.3. It should be noted that these figures reflect the new City of Tucson policy of planting (and irrigating) winter rye grass at municipal golf courses. Prior to this winter there was no winter irrigation requirement at the Randolph golf courses, thereby allowing sufficient effluent to maintain an 18 acre lake. The new irrigation requirement for winter rye grass leaves only enough effluent available to maintain an 8 acre lake.

The properties of the 8 acre lake that could be maintained under the existing conditions are shown on Table 4.4. Note that no groundwater is to be used to maintain the elevation of the water in the lake. However, some source such as groundwater will be required for the initial filling of the lake. Assuming a minimum depth of nine feet, it would require 61 acre-feet to fill this 8 acre lake.

Since the water entering the lake from the Randolph plant is available only in the winter the lake elevation will rise during this time. The lake elevation will lower in the summer due to the high evaporation rates. The monthly changes in the lake elevation are shown in Table 4.4. It can be seen from this table that the fluctuation between the maximum and minimum lake elevations is 6.0 feet.

If the funding can be obtained for the construction of the pipeline and lift station necessary to have the Randolph plant operating at full capacity, either the second situation (only the effluent in excess of the irrigation requirements for the golf courses will be available for the proposed lake) or the third situation (all of the additional effluent will be available for the proposed lake) can occur.

The second situation would result in the monthly water usages on Table 4.5. The properties of the 61 acre lake that could be maintained by the effluent in excess of golf course requirement are shown on Table 4.6. As with the smaller lake, this lake would have to be filled initially with some other source, such as groundwater. Assuming an average depth of nine feet, it would require 528 acre-feet to fill the lake. Once the lake was filled, the fluctuation between the maximum and minimum lake elevation would be only 3.7 feet. This is an improvement over the 6.0 feet that was calculated for the smaller lake.

The third situation would result in the monthly water usages shown on Table 4.7. The properties of the 98 acre lake that could be maintained by all of the additional effluent are shown on Table 4.8. The groundwater required to fill this lake would be 855 acre-feet, and the fluctuation in lake elevation over a period of one year would be only 1.7 feet.

CURRENT WATER USAGE (ACRE-FEET)													
MONTH	J	F	M	A	M	J	J	A	S	O	N	D	ANNUAL
Effluent Produced At Randolph	80	80	80	80	80	80	80	80	80	80	80	80	960
Water Requirements At Randolph Golf Courses	53.3	79.7	77.2	101.8	138.8	152.8	152.8	138.8	106.2	81.6	80.0	53.3	1216.3
Groundwater Pumped At Randolph	12.8	18.1	29.2	60.3	111.3	130.6	130.6	171.3	66.4	32.5	19.2	12.8	735.1
Water Requirement At Reid Park (Groundwater Only)	12.8	18.1	29.2	38.5	52.5	57.8	57.8	52.5	40.2	30.9	19.2	12.8	422.3
Excess Water Available For Ajo Wet Park	26.7	0.3	2.8	0	0	0	0	0	0	0	0	26.7	56.5

TABLE 4.3

CURRENT WATER USAGE:
SITUATION #1

	LOSS IN	EFFLUENT AC-FT	SURFACE AREA, AC	VOLUME AC-FT	DEPTH FT
JAN	4.3	26.7	8.0	107.1	15.0
FEB	2.7	0.3	7.9	105.6	14.8
MAR	2.7	2.8	7.9	106.7	15.0
APR	4.5	0.0	7.9	103.7	14.6
MAY	6.3	0.0	7.9	99.5	14.1
JUN	8.5	0.0	7.9	94.0	13.4
JUL	11.3	0.0	7.8	86.6	12.5
AUG	13.7	0.0	7.7	77.8	11.4
SEP	9.8	0.0	7.6	71.6	10.4
OCT	10.0	0.0	7.5	65.3	9.7
NOV	7.8	0.0	7.5	60.5	9.0
DEC	6.0	26.7	7.7	83.3	12.1

MINIMUM DEPTH, FT 9.0
 CALCULATED BOTTOM WIDTH, FT 361.8
 CALCULATED BOTTOM LENGTH, FT 723.5

TABLE 4.4

PROPERTIES OF
 8 ACRE LAKE

PROPOSED WATER USAGE (ACRE- FEET)													
MONTH	J	F	M	A	M	J	J	A	S	O	N	D	ANNUAL
Effluent Produced At Randolph	135	135	135	135	135	135	135	135	135	135	135	135	1620
Water Requirements At Randolph Golf Courses	53.3	79.7	77.2	101.8	138.8	152.8	152.8	138.8	106.2	81.6	80.0	53.3	1216.3
Groundwater Pumped At Randolph	12.8	18.1	29.2	38.5	56.3	76.5	76.5	56.3	40.2	30.9	19.2	12.8	735.1
Water Requirement At Reid Park *	12.8	18.1	29.2	38.5	52.5	57.8	57.8	52.5	40.2	30.9	19.2	12.8	422.3
Excess Water Available For Ajo Wet Park	81.7	55.3	57.8	33.2	0	0	0	0	28.8	53.4	55.0	81.7	446.9

* Groundwater Only

TABLE 4.5
WATER USAGE:
SITUATION #2

	LOSS IN	EFFLUENT AC-FT	SURFACE AREA, AC	VOLUME AC-FT	DEPTH FT
JAN	4.3	81.7	61.3	656.6	11.1
FEB	2.7	55.3	61.4	698.1	11.8
MAR	2.7	57.8	61.6	742.0	12.5
APR	4.5	33.2	61.7	752.1	12.7
MAY	6.3	0.0	61.6	719.7	12.1
JUN	8.5	0.0	61.5	676.2	11.4
JUL	11.3	0.0	61.3	618.4	10.5
AUG	13.7	0.0	61.1	548.7	9.4
SEP	9.8	28.8	60.9	527.7	9.0
OCT	10.0	53.4	60.9	530.3	9.0
NOV	7.8	55.0	61.0	545.7	9.3
DEC	6.0	81.7	61.1	596.9	10.1

MINIMUM DEPTH, FT 9.0
 CALCULATED BOTTOM WIDTH, FT 1109.7
 CALCULATED BOTTOM LENGTH, FT 2219.4

TABLE 4.6
 PROPERTIES OF
 61 ACRE LAKE

PROPOSED WATER USAGE (ACRE-FEET)													
MONTH	J	F	M	A	M	J	J	A	S	O	N	D	ANNUAL
Effluent Produced At Randolph	135	135	135	135	135	135	135	135	135	135	135	135	1620
Water Requirement At Randolph Golf Courses	53.3	79.7	77.2	101.8	138.8	152.8	152.8	138.8	106.2	81.6	80.0	53.3	1216.3
Groundwater Pumped At Raldolph	12.8	18.1	29.2	60.3	111.3	130.6	130.6	111.3	66.4	32.5	19.2	12.8	735.1
Water Requirement At Reid Park *	12.8	18.1	29.2	38.5	52.5	57.8	57.8	52.5	40.2	30.9	19.2	12.8	422.3
Excess Water Avail- able For Ajo Wet Park	81.7	55.3	57.8	55.0	55.0	55.0	55.0	55.0	55.0	55.0	55.0	81.7	716.5

* Groundwater Only

TABLE 4.7
WATER USAGE:
SITUATION #3

	LOSS IN	EFFLUENT AC-FT	SURFACE AREA, AC	VOLUME AC-FT	DEPTH FT
JAN	4.3	81.7	98.0	934.3	9.8
FEB	2.7	55.3	98.1	967.6	10.2
MAR	2.7	57.8	98.2	1003.3	10.5
APR	4.5	55.0	98.3	1021.4	10.7
MAY	6.3	55.0	98.4	1024.8	10.7
JUN	8.5	55.0	98.4	1010.1	10.6
JUL	11.3	55.0	98.3	972.5	10.2
AUG	13.7	55.0	98.2	915.4	9.6
SEP	9.8	55.0	98.0	890.3	9.4
OCT	10.0	55.0	98.0	863.7	9.0
NOV	7.8	55.0	97.9	855.1	9.0
DEC	6.0	81.7	97.9	887.8	9.3

MINIMUM DEPTH, FT 9.0
 CALCULATED BOTTOM WIDTH, FT 1418.1
 CALCULATED BOTTOM LENGTH, FT 2836.2

TABLE 4.8
 PROPERTIES OF
 98 ACRE LAKE

ALTERNATE SOURCES:

Two other sources that were considered for use at the Ajo Wet Park were storm water and Tucson Electric Power Company blowdown water.

Storm water was determined to be an undesirable source since it arrives in large quantities at infrequent intervals. This would require a tertiary treatment plant with a large capacity. This would be very uneconomical since it would remain idle most of the year.

The blowdown water also proved to be an unreliable source. As the Tucson Electric Power Company transitions from natural gas and oil powered generating stations to coal fired generating stations, there will be less and less cooling tower water available. Also, additional money would be required for modifications at the generating plant and for construction of a pipeline between the generating plant and Ajo Wet Park.

PART 5

ADDITIONAL TREATMENT

There are various alternative tertiary processes that can be used on the Randolph Park effluent to bring it up to the required water quality standards. Some of these processes and their efficiency in treating secondary effluent are listed in Table 5.1.

It was desired to keep the cost of the additional treatment and the difficulty of operation to a minimum. Therefore, expensive methods of treatment, such as reverse osmosis and ion exchange, were not considered as feasible options. The processes that were considered are land treatment, filtration, microscreening, carbon adsorption, phosphorus removal, nitrogen removal, and polishing ponds.

LAND TREATMENT:

The three major types of land treatment are irrigation, overland flow, and infiltration-percolation. Since irrigation and overland flow typically have low recovery rates (80%) they won't be considered further. With a well developed recovery technique up to 97% of the effluent treated by infiltration-percolation can be recovered.

Infiltration-percolation treats wastewater with a minimum of land area and at a high rate. The maximum effluent that the land would be required to handle occurs in December and January. During these two months a loading rate of close to 0.5 mgd would be required. If an application rate of 0.15 ft/day is used the land area required for this treatment would be approximately 10 acres. During the summer when there is no effluent to treat, the soil would be able to rest, which would restore its infiltration and treatment capacity.

The two methods that could be used in recovering the renovated water are pumping with wells or gathering it in underdrains. These are illustrated in Figure 5.1.

FILTRATION:

Granular media filtration and microscreening are normally used when a reduction in suspended solids and BOD is desired. The major granular media filter configurations are shown in Figure 5.2. The down-flow filters as shown in Figure 4.2, (a), (b) and (d) require back-wash water that averages about 5 percent of the flow.

Outdoor sand beds can also be used to treat the effluent. The effluent is placed on one bed for 24 hours. The bed is then drained and allowed to dry for one to two days. By intermittently operating three beds in this fashion a three day cycle can be established. This cycle can produce a high quality effluent. For a loading of

CAPABILITIES OF WASTEWATER TREATMENTS									
TREATMENT TYPE	BOD Mg/l*	%Removal	TSS Mg/l	%Removal	SS Mg/l	%Removal	NO ₃ Mg/l	%Removal	PO ₄ Mg/l
Primary & Secondary**									
Waste Stabilization Lagoon	30-60	70			30-60	70	15	20-30	
Extended Aeration	20-20	80-90			10	95	15	20-30	10-20
Primary Sedimentation	120	45			75	70			
High Rate Trickling Filters	40	80			20-30	90	15	20-30	10-20
Standard Rate Trickling Filters	20-30	85			10-12	95	10	30-40	5
High Rate Activated Sludge	30-50	75			20-25	90	10	20-30	10-20
Standard Rate Activated Sludge	15-20	90			20-25	90	10	30-40	8
Physical-Chemical	10-15	93		20	10-20	90	4.6	75	1.5
Tertiary***									
Chemical Coagulation & Filtration	4-12	94-98			5-7	96-98			98
Sand-Filtration - Deep Bed	4-12	94-98			5-7	96-98			98
Microscreening	4-12	94-98			2-6	97-99			
Carbon Adsorption	2-10	95-99			1-3	98			
Microbial Denitrification							4	60-95	
Ammonia Stripping/B.P. Chlorination (10 mg Cl per 1.0 mg NH ₃)	1-3	98					4	85-98	
Ion Exchange				<1	99+		5	80-90	98+
Land Disposal/Ground Drains	1-2	99			0-2	99	17	5-15	0.5-1.0
Reverse Osmosis	1-2	99			<1	99+	<1	95-99	<1.0
Electrodialysis	1-2	99		40			10	30-50	6
* = Mg/l of constituent remaining in effluent ** = Values based on typical raw sewage influent *** = Overall effluent quality and removals when process is preceded by primary and secondary treatment									
SOURCE: Bishop, Grenney, Narayanan, and Klemetsen: Evaluating Water Reuse Alternatives in Water Resources Planning. Utah Research Laboratory, College of Engineering, Utah State University, 1974.									

TABLE 5.1

CAPABILITIES OF
WASTEWATER TREATMENTS

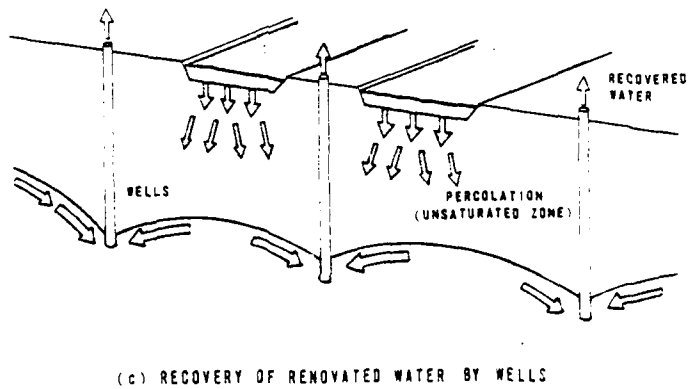
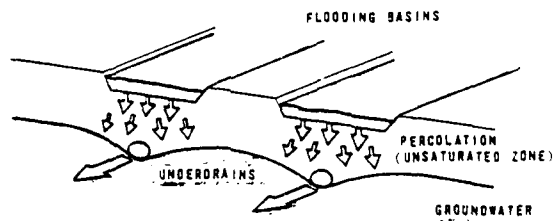
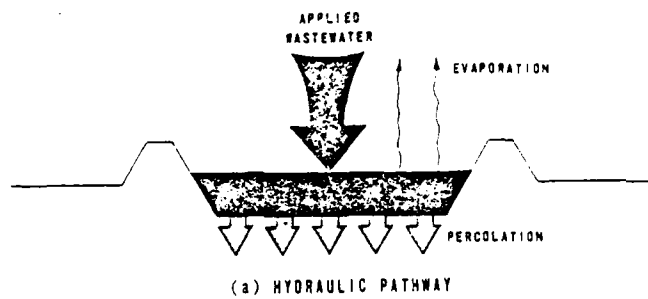


FIGURE 5.1
RAPID INFILTRATION

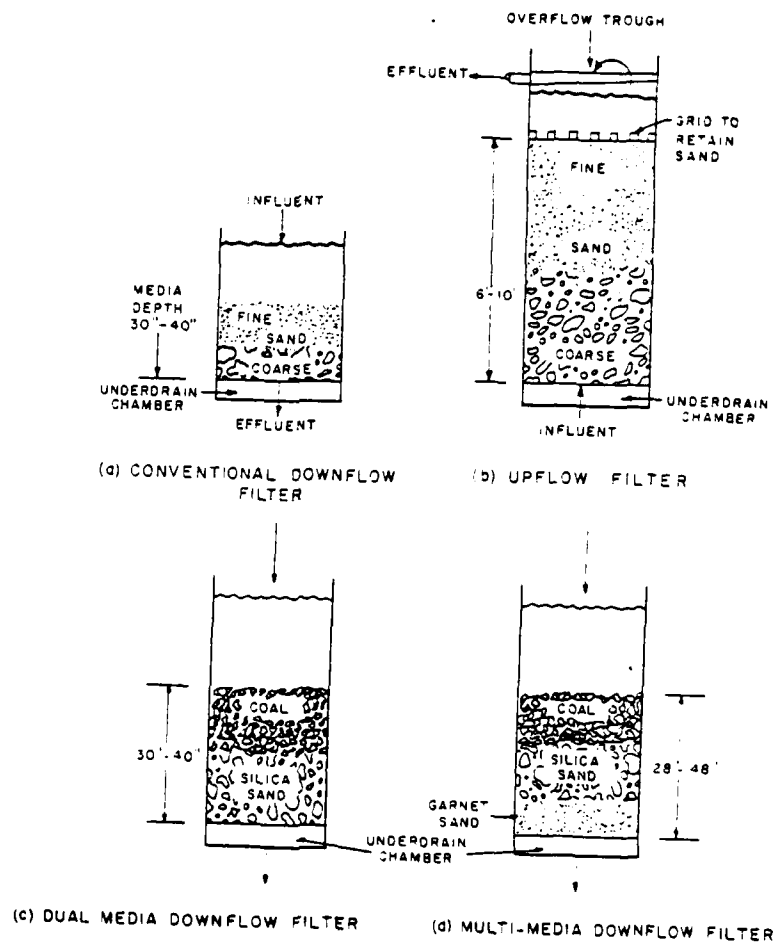


FIGURE 5.2
FILTER CONFIGURATIONS

0.5 mgd a sand bed size of approximately one acre would be required.

A microscreening unit such as is shown in Figure 5.3 is often used to polish secondary effluent. However, such units are more expensive in capital outlay and operating costs than the sand filters. The microscreens also cannot produce the high quality that can be obtained with a sand filter.

POLISHING PONDS:

Polishing ponds can reduce the settleable solids, BOD, and fecal coliforms. If the retention time is greater than three days then a reduction in phosphorus can also be obtained. However, there is a gain in suspended solids as the retention time increases. The relationship between various components of the wastewater and the retention time is shown in Figure 5.4.

The required size of the pond is based upon the daily load it is designed to treat. The following calculations would be typical for the Randolph plant effluent:

$$\text{Average BOD}_5 = 15 \text{ mg/l}$$

$$\begin{aligned} \text{Average Flow} &= 500,000 \text{ gal/day} \\ &(\text{December, January}) \end{aligned}$$

$$\begin{aligned} \text{Daily Loading} &= 15 \text{ mg/l} \times \frac{1 \text{ lb.}}{453,592 \text{ mg}} \times 3.78543 \frac{\text{l}}{\text{gal}} \times 500,000 \text{ gal/day} \\ &= 63 \text{ lb/day} \end{aligned}$$

$$\text{Typical Loading on Tertiary Ponds} = 15 \text{ lb/day-acre}$$

$$\text{Size of Pond} = 63/15 = 4.2 \text{ acres}$$

Fish could be introduced into the polishing pond which would help reduce the suspended solids. This pond could then be used as a non-boating fishing area.

To obtain a higher quality effluent, a rock filter, as shown in Figure 5.5, could be added. A biological film forms on the rocks which helps to filter the wastewater. Because this film is anaerobic, a post aeration facility is required. This could possibly take the form of a cascade which would enhance the aesthetics of the lake.

CARBON ADSORPTION:

This process consists of passing the secondary effluent through a tank filled with carbon granules or carbon slurry. The impurities are adsorbed by the carbon provided a sufficient retention time is allowed. When the adsorption capacity of the carbon is exhausted it must be replaced or regenerated.

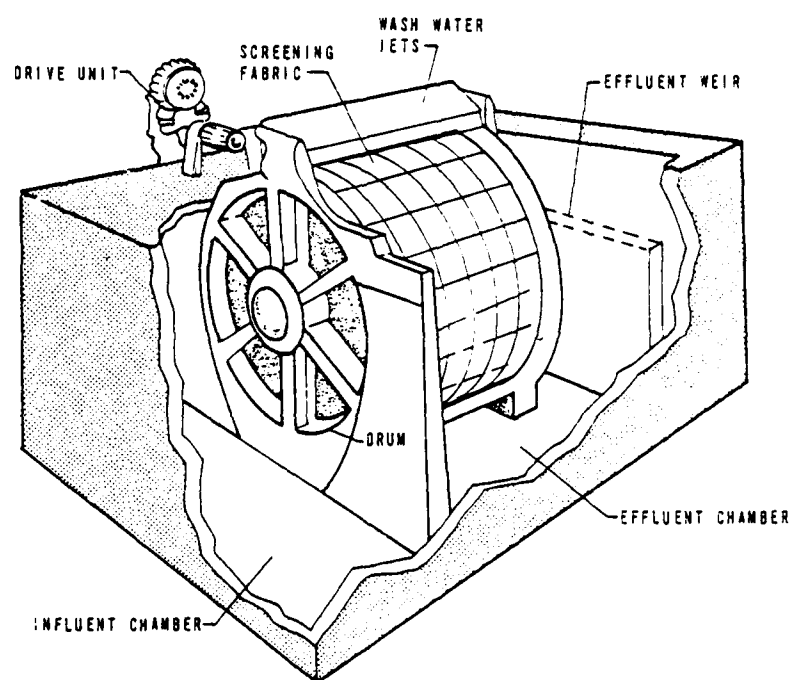


FIGURE 5.3
TYPICAL MICROSCREEN UNIT

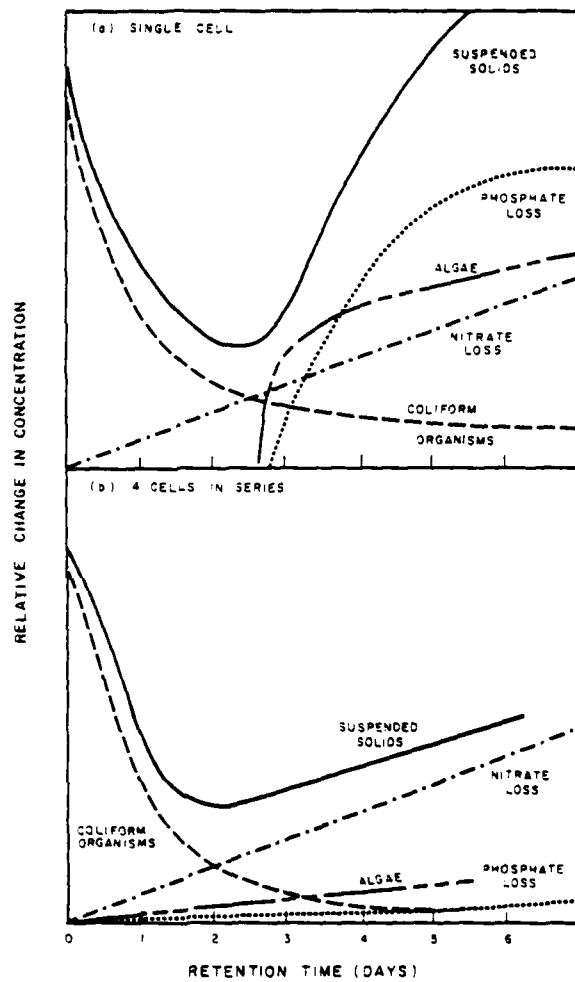


FIGURE 5.4

PERFORMANCE OF POLISHING
PONDS FOLLOWING SECONDARY
TREATMENT

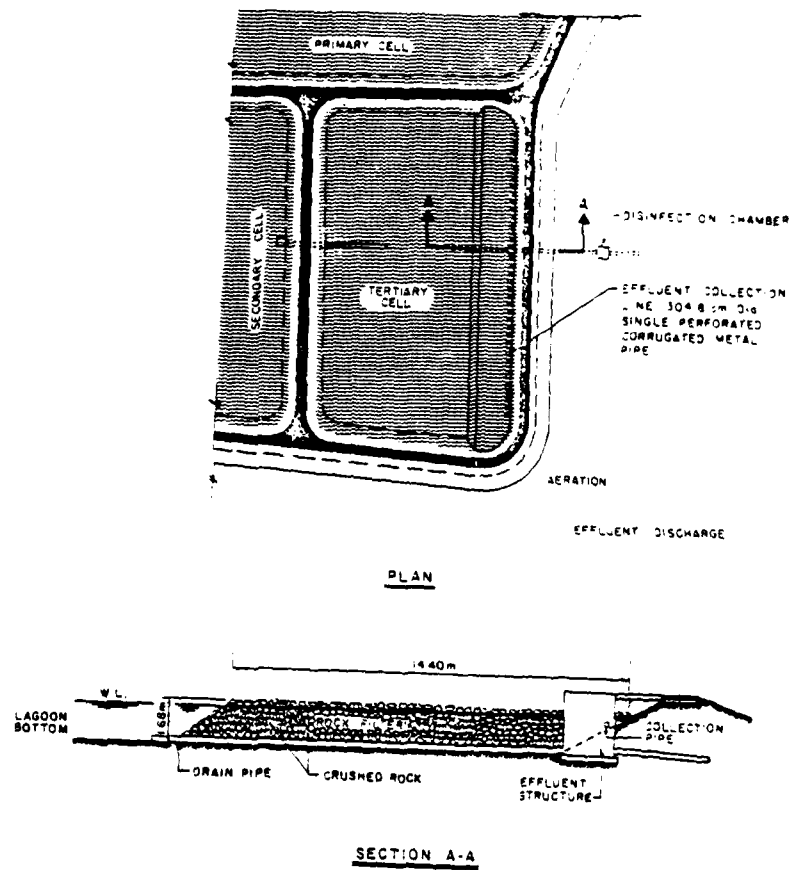


FIGURE 5.5
SUBMERGED ROCK FILTER

The equipment that is required for this process consists of vessels in which the carbon is held, piping, screens, and a regeneration furnace. This expensive equipment results in a high initial investment. It is also possible that an operator would be required at the tertiary plant on a daily basis. For these two reasons, this type of treatment is an unattractive alternative.

PHOSPHORUS REMOVAL:

Phosphorus can be removed from the secondary effluent by the addition of lime or inorganic salts. These chemicals combine with the phosphorus to form solids which settle out of the wastewater. In addition to phosphorus removal, the addition of chemicals can result in a significant removal of organic material. If settling alone is used, the residual phosphorus can be reduced to 1.0 mg/l. If higher removals are desired, filtration must be used.

NITROGEN REMOVAL:

There are two general types of methods to remove nitrogen from wastewater. The first type encompasses the physical chemical methods such as ionexchange, ammonia stripping, and break point chlorination. These methods, with the possible exception of breakpoint chlorination, are usually expensive.

The second type of nitrogen removal is the biological method. This consists of conversion of the nitrogen in the form of ammonia to nitrogen in the form of nitrate. This is accomplished by introducing oxygen into the effluent. If the quantity of the resulting nitrate is unacceptably too high, then denitrification must be used. The denitrification process converts the nitrate into nitrogen gas which is allowed to escape to the atmosphere.

PART 6

FINDINGS AND CONCLUSIONS

In general it has been found that the effluent from the Randolph Wastewater Treatment Plant is deficient in both quantity and quality for the filling and maintenance of a recreational lake without extensive capital improvements to the sewage collection system and the treatment plant. The problem areas are summerized as follows:

- A. The sewage collection system feeding the Randolph Wastewater Treatment Plant cannot deliver sufficient sewage for the plant to operate at its design capacity.
- B. The maximum lake size that can be maintained by the existing system is 8 acres.
- C. If the sewage collection system were to be improved, the quantity of effluent treated at Randolph would be sufficient to maintain a lake ranging in size from 61 to 98 acres, depending upon the quantity of effluent retained for irrigational purposes at Randolph.
- D. Regardless of lake size, a supplemental source of water will be required for the initial filling of the lake, as well as for domestic uses of the proposed wet park. Based upon the park visitor estimates shown on Table 6.1, and assuming 12 hour daily operation, and an average water demand of 15 gallons per person per day, it was estimated that the proposed park would require a 200 gpm well. In addition, the following estimates were made:
 1. For an 8 acre lake.

Daily visitors	= 7,928 persons
Domestic water use	= 118,920 gallons per day
Water needed to fill lake	= 61 acre-ft.
Time to fill lake with effluent plus groundwater	= 35 days.
 2. For a 61 acre lake.

Daily visitors	= 9,282 persons
Domestic water use	= 139,230 gallons per day
Water needed to fill lake	= 528 acre-ft.
Time to fill lake with effluent plus groundwater	= 166 days.
 3. For a 98 acre lake.

Daily visitors	= 10,022
Domestic water use	= 150,330 gallons per day

ANNUAL RECREATIONAL DAY EXPECTED

Activity	Unit	Density	Daily Turnover	Annual Recreation Days
Lake Boating & Fishing	70 ac.	6 persons/ac.	2	84,000
Lake Shore Fishing	7,000 L.F.	1/25 persons/L.F.	4	112,000
Information Center	5 ac.	25 persons/ac.	10	125,000
Playgrounds/ Fields	13.3 ac.	75 persons/ac.	4	400,000
Hiking, walking & jogging trails	50 ac.	3 persons/ac.	10	150,000
Picnicking	70 tables	5 persons/table	2	70,000

TOTAL 941,000

Source: Pima County Department of Parks and Recreation

TABLE 6.1

ANNUAL RECREATIONAL
DAY EXPECTED

Water needed to fill lake = 855 acre-ft.

Time to fill lake with effluent plus groundwater = 296 days.

- E. The quality of the effluent from the Randolph Wastewater Treatment Plant is not high enough for its use in a lake for fishing and boating. The effects of various constituents in water (and effluent) upon freshwater organisms is dependent upon many variables such as species, age of the organism, temperature, and the cumulative synergistic and antagonistic effects of toxicants present. For example, zinc has been found to be toxic at concentrations as low as 0.1 mg/l (zinc forms insoluble compounds with the mucous film on fish gills) but the presence of copper increases the overall toxicity (synergism) while the presence of calcium decreases the toxic effects (antagonism) (6). Due to this great array of variables it is impossible to prescribe an all encompassing list of water quality criteria for aquatic organisms. The following general criteria, however, can be presented as applicable to the Randolph Wastewater Treatment Plant effluent: (6) (15)

1. Dissolved materials: not greater than 50 milliosmoles (equivalent to 1500 mg/l of NaCl)
2. pH: in the range of 6.0 - 9.0
3. Alkalinity: not less than 20 mg/l
4. Dissolved Oxygen: not less than 5 mg/l
5. Carbon Dioxide: not greater than 25 mg/l
6. Turbidity: not greater than 25 Jackson units
7. Total phosphorus, (P): not greater than 0.05 mg/l
8. Total nitrogen, (N): not greater than 10 mg/l
9. Ammonia, (N): not greater than 1.5 mg/l
10. Iron: not greater than 0.2 mg/l
11. Lead: not greater than 0.1 mg/l
12. Zinc: not greater than 0.1 mg/l
13. Copper: not greater than 0.02 mg/l

Comparison of the effluent quality (Table 3.1) to these criteria reveals the following problems:

1. Dissolved oxygen content is too low
2. Turbidity is too high
3. Phosphorous content is too high
4. Ammonia content is too high
5. Iron, zinc and copper content is too high

Additional laboratory testing and pilot treatment plant operations must be performed in order to establish, positively, the proper stages of advanced treatment and the optimum chemical dosages required to make the effluent suitable for aquatic life. A preliminary estimate of the additional treatment required indicates that two stage tertiary lime treatment (for phosphorous and heavy metals removal) followed by ammonia stripping, and filtration (for reduction of turbidity) will be required. Discriptions of these processes, and the methodology used to estimate capital and annual costs are found in Appendix A.

- F. The estimated costs were calculated using the cost equations found in Appendix A with the following modifications:
1. Base capital costs were multiplied by the ratio of the projected Engineering News Record Construction Cost Index of 3425 for 1980 to the index of 1850 for February, 1973.
 2. The cost of land was assumed to be \$5,000 per acre.
 3. The interest rate used was 8%.
 4. The wage rate used was \$7.20 per hour.
 5. Base material costs were multiplied by the ratio of the projected Engineering News Record materials index of 850 for 1980 by the index of 488 for February, 1973.

The estimated costs that are presented here were calculated utilizing equations that were empirically derived from curves of cost data versus plant size (see Appendix A) and are extremely rough. These costs are to be used for very preliminary planning purposes.

1. 0.5 mgd raw sewage pumping station to increase influent flow to Randolph Wastewater Treatment Plant.
 Capital Cost = \$431,576

Amortized Construction Cost	=	14.02 ¢ per 1000 gallons
Fixed O & M cost	=	3.00 ¢ per 1000 gallons
Variable O & M cost	=	0.53 ¢ per 1000 gallons
Annual Cost	=	17.55 ¢ per 1000 gallons
Annual Cost	=	\$32,029 per year

2. 0.81 mgd two stage tertiary lime treatment.
 Capital Cost = \$1,446,487
 Amortized Construction Cost = 46.99 ¢ per 1000 gallons
 Fixed O & M Cost = 11.01 ¢ per 1000 gallons
 Variable O & M Cost = 1.76 ¢ per 1000 gallons
 Annual Cost = 69.76 ¢ per 1000 gallons
 Annual Cost = \$218,722 per year

3. 0.81 mgd ammonia stripping unit.
 Capital Cost = \$178,233
 Amortized Construction Cost = 5.79 ¢ per 1000 gallons
 Fixed O & M Cost = 15.66 ¢ per 1000 gallons
 Variable O & M Cost = 4.89 ¢ per 1000 gallons
 Annual Cost = 26.34 ¢ per 1000 gallons
 Annual Cost = \$82,585 per year

4. 0.81 mgd filtration unit.
 Capital Cost = \$850,223
 Amortized Construction Cost = 27.62 ¢ per 1000 gallons
 Fixed O & M Cost = 2.89 ¢ per 1000 gallons
 Variable O & M Cost = 14.26 ¢ per 1000 gallons
 Annual Cost = 44.77 ¢ per 1000 gallons
 Annual Cost = \$140,370 per year

APPENDIX A

**EXCERPTS FROM
"A GUIDE TO THE SELECTION OF
COST-EFFECTIVE WASTEWATER
TREATMENT SYSTEMS"**

AB. RAW WASTEWATER PUMPING

Raw wastewater pumping stations are normally designed to provide sufficient hydraulic head to permit gravity flow through the treatment plant. In general, it has been found to be more cost-effective to provide sufficient head in a single pump station than to provide multiple stations of lower heads within the plant. The exceptions to this are those situations where site topography prevents plant layout in such a manner that gravity flow can be readily achieved.

PUMP STATION DESIGN

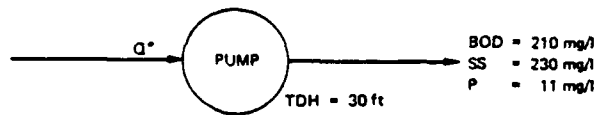
Pump station design varies widely from plant to plant because of differences in capacity and head requirements. Generally, these pumping stations are of relatively low head (10'-40') and high capacity. Various types of pumps are in common use, but probably the most popular are the open impeller centrifugal pumps. Although these are relatively inefficient, they offer the advantage of being able to pass large solids, an essential feature of pumps used in this capacity.

An important consideration in the design of raw waste pumping stations is the diurnal variation of wastewater flow. Because of this variation, the pumping station must be capable of handling the peak instantaneous flow during the life of the plant. Further, sufficient standby capacity must be provided to insure that the maximum flow can be handled even though a portion of the pumping facility is out of service.

For this study, pump capacities were assigned for estimation of capital and O & M based on the 24 hour peak flow, while costs of the pumping station structure were based on ultimate pumping capacity. A typical value of 30' TDH (total dynamic head) was used to determine horsepower requirements and power costs.

AB. RAW WASTEWATER PUMPING

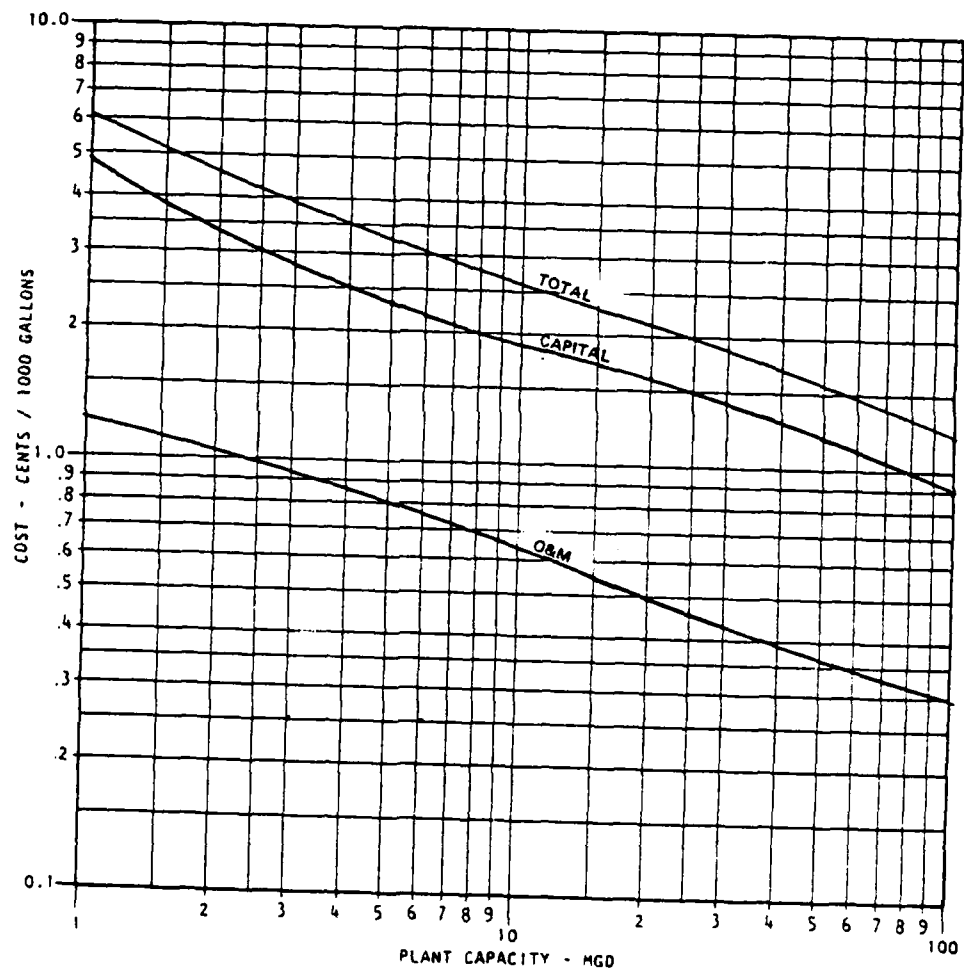
Influent: Effluent from Preliminary Treatment AA



*Peak Capacity (with largest unit out of service) = $2 \times Q$

Design criteria in bold type are per
mgd influent wastewater flow.

AB. RAW WASTEWATER PUMPING
Influent: Effluent from Preliminary Treatment AA



F. TWO-STAGE TERTIARY LIME TREATMENT

This process is very similar in purpose to primary sedimentation. With two-stage lime addition, however, it follows, rather than precedes, biological treatment.

Reasons for placing lime treatment after biological treatment include:

- o additional BOD removal
- o phosphorous removal
- o lime sludge produced is relatively uncontaminated with organic matter

The reader is directed to the discussion of Primary Sedimentation with Two-Stage Lime Addition, unit process A2, for a more complete description and other information pertaining to efficiency and cost.

A2 TWO STAGE LIME ADDITION

In this modification, lime is rapidly mixed with the first-stage influent to obtain a minimum pH of 11. The water is stirred in the first-stage up-flow solids contact clarifier to encourage floc formation and precipitation. Following settling the water is discharged from the first-stage unit. Carbon dioxide is added to the water to reduce the pH in the second-stage to approximately 10, within range of the minimum calcium carbonate solubility. The water is again flocculated, settled and discharged. The pH of the process effluent is then neutralized with CO₂ or acid.

Solids produced in the first and second-stage units are mechanically collected, thickened and pumped to sludge treatment processes.

The increased efficiency of this process over conventional primary sedimentation resulting from flocculation of small suspended particles and precipitation of the oxygen demanding material combined with other physical chemical unit operations, sometimes eliminates the need for biological treatment. As much as 80% of the influent BOD, more than 90% of the suspended solids and approximately 90% of the phosphorus can be removed from the raw wastewater by this process. If a higher quality effluent is required this process can be followed by filtration and activated carbon to produce an effluent significantly lower in BOD and suspended solids than conventional secondary effluent.

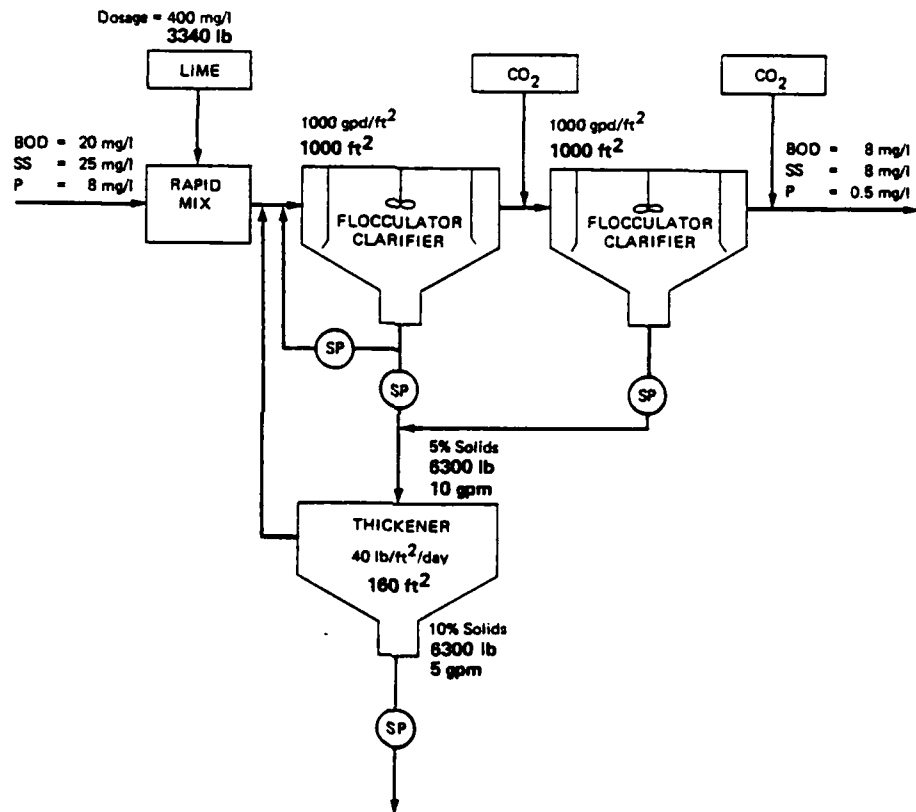
Advantages of using lime over other chemical agents include

- 1) more easily dewaterable sludge solids,
- 2) almost complete destruction of bacteria and viruses,
- 3) precipitation of nearly all heavy metals, and
- 4) potential for recovery of much of the lime used.

A major disadvantage is the handling and disposal of large quantities of lime sludge.

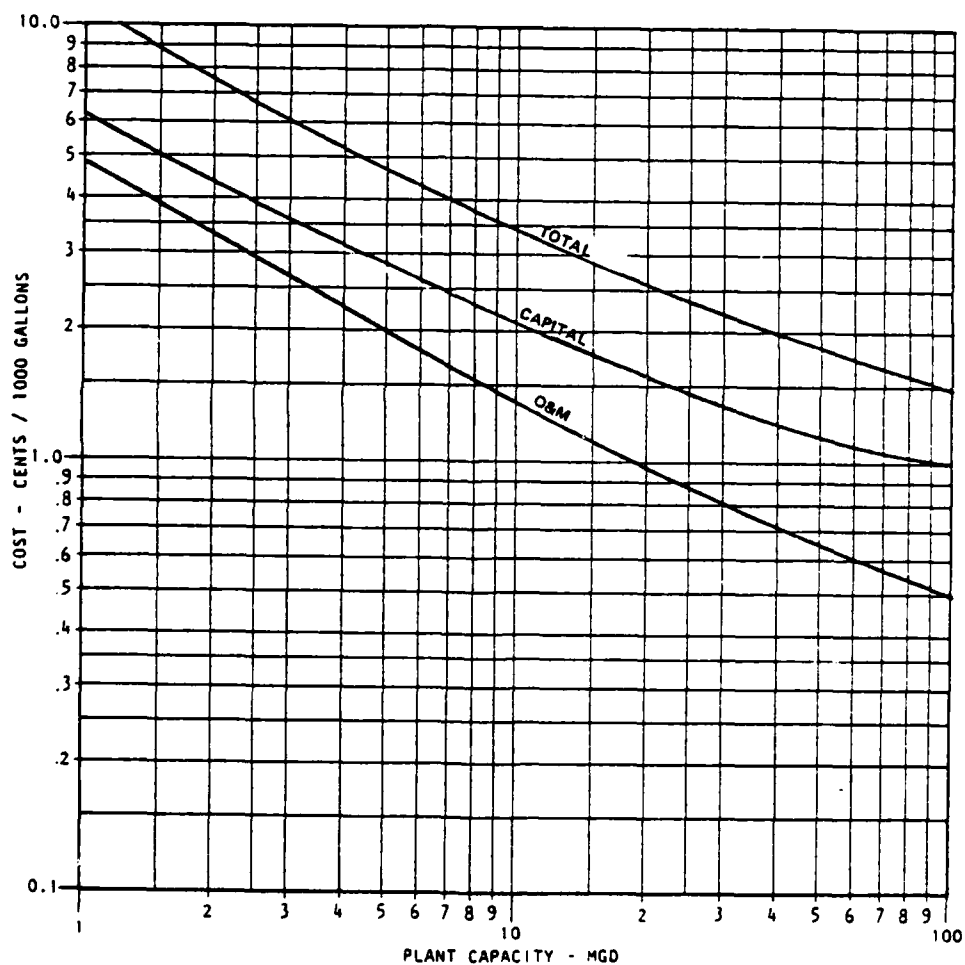
F-2. TWO-STAGE TERTIARY LIME TREATMENT

Influent: Effluent from Conventional Activated Sludge C-1



Design criteria in bold type are per
mgd influent wastewater flow.

F-2. TWO-STAGE TERTIARY LIME TREATMENT
 Influent: Effluent from Conventional Activated Sludge C-1



K. AMMONIA STRIPPING

Ammonia may be removed from wastewater by a physical stripping process. If the pH of a wastewater is raised to about 11 essentially all of the ammonium ion is converted to free ammonia. This gas may then be stripped by passing the waste through a packed tower having a counter current flow of air.

Stripping towers can be very effective in ammonia removal but their efficiency is highly dependent on air temperature. As the air temperature decreases, the ammonia removal efficiency drops significantly. This process, therefore, is not recommended in a cold climate. A major operational disadvantage of stripping is calcium carbonate scaling in the tower, which has been a persistent problem. Furthermore, the discharge of ammonia to the atmosphere can be a significant source of air pollution.

ITEMS INCLUDED IN COST ESTIMATES

Capital Cost

The major pieces of equipment included in the capital cost are listed below.

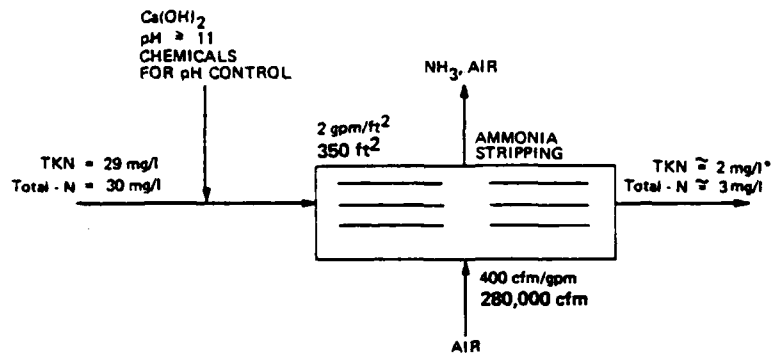
1. Ammonia stripping towers
2. Blower

O&M Costs

Costs are divided between power and labor.

K. AMMONIA STRIPPING

Influent: Effluent from First Stage of Two Stage Tertiary Lime Treatment F-1 or F-2

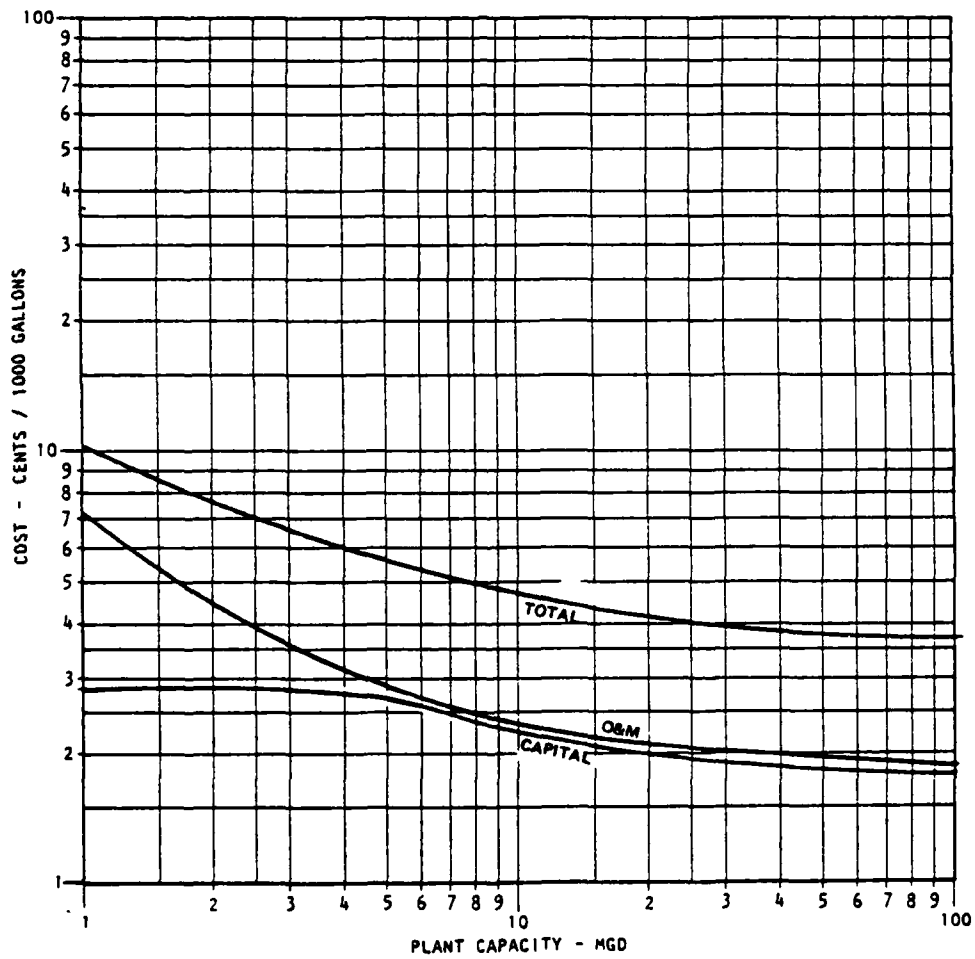


*90% removal efficiency at 80°F ambient air temperature

Design criteria in bold type are per
mgd influent wastewater flow.

K. AMMONIA STRIPPING

Influent: Effluent from First Stage of Two Stage Tertiary Lime Treatment F-1 or F-2



D. GRAVITY FILTRATION

All of the suspended matter in wastewater cannot be removed by gravity settling even after coagulation and flocculation. Thus, if a low effluent suspended solids concentration is required, filtration of the biological or two stage lime treatment effluent is generally necessary.

Filtration consists of passing the wastewater through a bed of porous material, separating the suspended matter from the water. As solids accumulate in the filter bed, it becomes necessary to backwash the bed by passing clean water at a high rate through the filter in a reverse direction to that of normal flow. The wash water, containing the suspended solids, is generally returned to the head of the plant and recycled through the primary clarifiers, although the wash water optionally can be returned to the other plant clarifiers when appropriate.

ITEMS INCLUDED IN COST ESTIMATES

Capital Cost

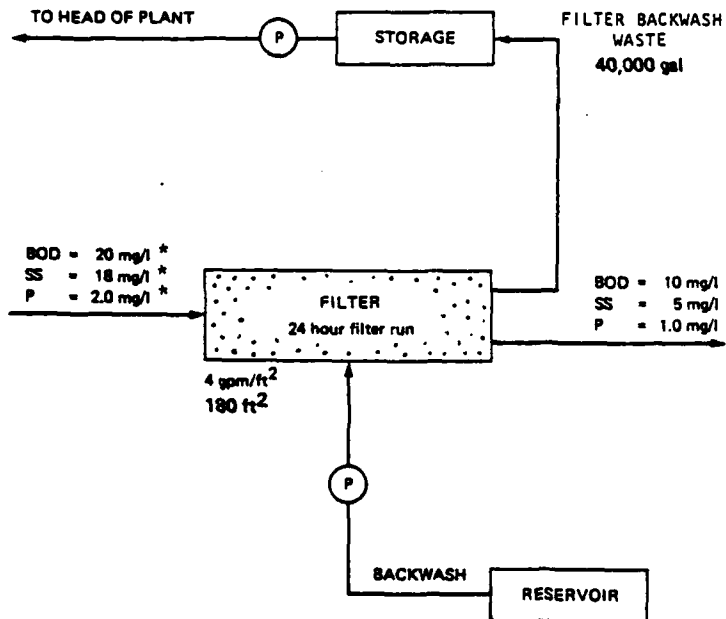
The capital cost of a filter installation is a function of the loading rate at which the wastewater may be applied. This loading is determined by characteristics of the filter media and the solids level of the water being applied. For this study a filtration rate of 4 gal/min/ft² was assumed, which is satisfactory for either a high quality biological effluent or effluent from two stage lime treatment. Facilities for storage of backwash water and all requisite pumps and piping were included.

O&M Costs

The O&M costs include all power and labor associated with filtration and backwash cycles.

D. FILTRATION

Influent: Effluent from Primary Sedimentation - Two Stage Lime Addition
 A-2
 Activated Sludge - Alum or FeCl_3 Addition
 C-4 or C-5
 Two Stage Tertiary Lime Treatment
 F-1 or F-2
 Trickling Filter B-2, B-3
 Activated Sludge C-2, C-3
 Biological Nitrification G-1, G-2, G-3, G-4
 Biological Denitrification H
 Breakpoint Chlorination J
 Ammonia Stripping K



*Averaged values from above influent.

Design criteria in bold type are per
 mgd influent wastewater flow

D. FILTRATION

Influent: Effluent from Primary Sedimentation — Two Stage Lime Addition

A-2

Activated Sludge — Alum or FeCl_3 Addition

C-4 or C-5

Two Stage Tertiary Lime Treatment

F-1 or F-2

Trickling Filter B-2, B-3

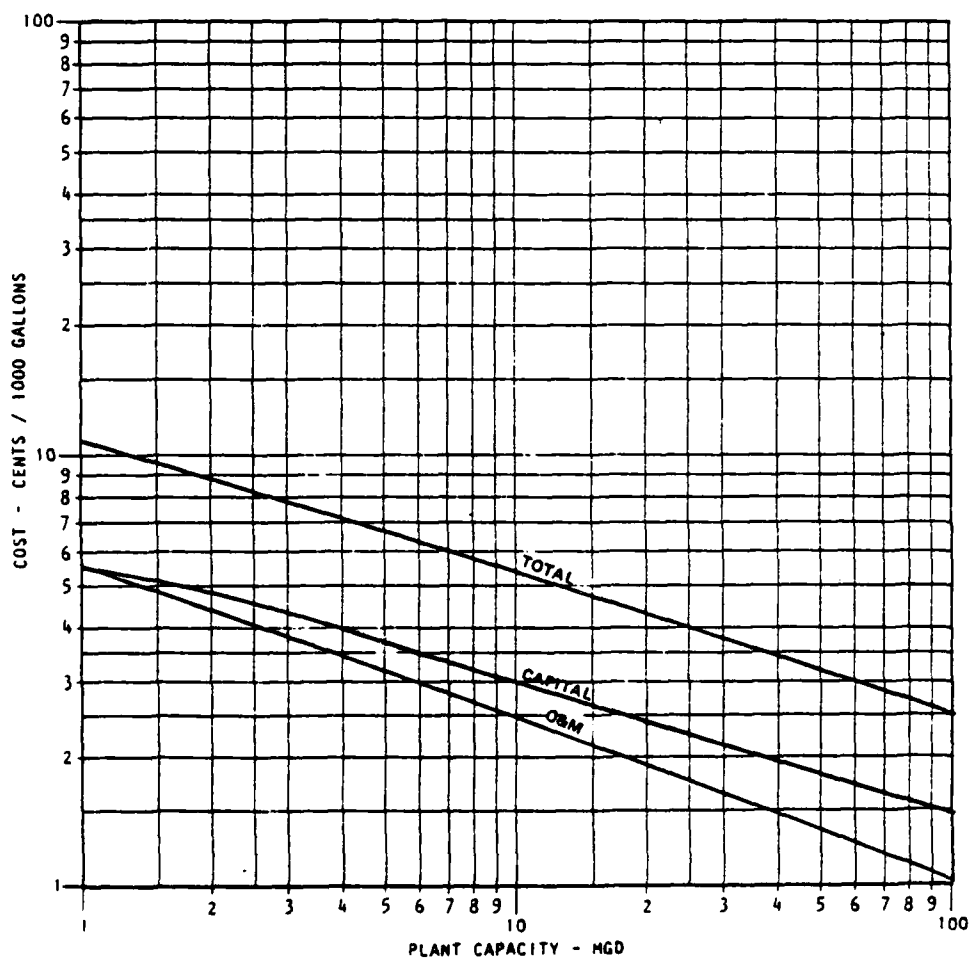
Activated Sludge C-2, C-3

Biological Nitrification G-1, G-2, G-3, G-4

Biological Denitrification H

Breakpoint Chlorination J

Ammonia Stripping K



COST EQUATIONS

Three formulae have been developed for each unit process. They are:

1. A formula for amortized capital cost
2. A formula for fixed operation and maintenance costs
3. A formula for flow-variable operation and maintenance cost

The formulae have been developed in such a manner that the following variables may be changed as time and/or specific conditions warrant:

<u>Variable</u>	<u>Units</u>	<u>Symbol</u>	<u>Value Used to Determine Cost Curves⁽¹⁾</u>
Plant capacity	MGD	Q	1, 5, 20, 100
Amortization Period	years	n	20
Interest Rate ⁽⁵⁾	%	i	5-5/8
Service and Interest Factor ⁽²⁾	%	SIF	27
Labor Rate	\$/man-hour	MHR	5
Land Cost	\$/acre	ULC	2000
Wholesale Price Index Industrial Commodities ^{(3), (5)}	—	WPI	120
National Average Wastewater Treatment Plant Cost Index ^{(4), (5)}	—	STP	177.5

(1) Refers to "A Guide to the Selection of Cost-Effective Wastewater Treatment Systems."

(2) This includes allowance for engineering, contingencies, and interest during construction.

(3) As of February 1973.

(4) As of February 1973.

(5) For April 1975, WPI = 169.7 STP = 232.5
and i = 5 7/8% (Water Resources Council).

Two expressions used commonly deserve a brief explanation.

The term $\frac{1}{3650 Q}$ appears in each of the three cost formulae and is

used to convert dollars per year into cents per thousand gallons. The

term $\frac{i(1+i)^n}{(1+i)^n - 1}$ appears in the formula for total amortized cost. It is

known as the capital recovery factor and converts total capital investment in dollars to a yearly amortized cost in dollars per year based on interest rate i and amortization period n .

Several other terms appear commonly throughout each group of formulae.

They are (1) base capital cost, BCC, (2) land requirements, LR, (3) base manhours, BMH, and (4) base material cost, BMC. Each term varies with flow for each unit process. Besides appearing individually in each group of unit process cost formulae, they are summarized in Table III-I appearing at the end of this section.

Note that the cost formulae have been developed using a least squares curve fitting method. The equations that appear in this section represent the best choice of equation over the range of input values. This is important to know when using the cost formulae for very small values of Q . In these situations, some equations yield negative values. This is true, for instance, with the formula for the land requirement for the ion-exchange process:

$$LR = -0.17 + 0.021 Q$$

For values of Q less than 8, the value of LR is less than 0.

The reader should realize that this occurs only due to poor curve fit at low values of Q . In such occurrences, a zero value should be substituted for any negative numbers thus obtained. Under no circumstances should a negative value be used.

The following formulae apply to any of the unit processes for which flow sheets were developed in Section II.

Total Amortized Capital Cost, ¢ / 1000 Gal. =

$$\left[\left(\text{BCC} \right) \left(\frac{\text{STP}}{177.5} \right) + \left(\text{LR} \right) \left(\text{ULC} \right) \right] \left[\frac{100 + \text{SIF}}{100} \right] \left[\frac{1}{3650Q} \right] \text{CRF}^{(1)}$$

where

BCC, Base Capital Cost, \$ = Refer to Table B-1
 LR, Land Requirement, Acres = Refer to Table B-1

Fixed Operation and Maintenance Costs, ¢ / 1000 Gal. =

$$\left(\text{BMH} \right) \left(\text{MHR} \right) \left(\frac{1}{3650Q} \right),$$

where

BMH, Base Man-hour Requirement, Man-hours/Yr. = Refer to Table B-1

Variable Operation and Maintenance Costs, ¢ / 1000 Gal. =

$$\left(\text{BMC} \right) \left(\frac{\text{WPI}}{120} \right) \left(\frac{1}{3650Q} \right),$$

where

BMC, Base Material Costs, \$/Yr. = Refer to Table B-1

(1) $\text{CRF (Capital recovery factor)} = \frac{i(1+i)^n}{(1+i)^n - 1}$

Table B-1
FLOW VARIABLE COST ELEMENTS

Unit Process	Base Capital Cost (BCC)	Land Requirement(LR)	Base Manhours (BMH)	Base Materials Cost (BMC)
AA	$32331 Q^{0.61}$	0	$1379.2 + 143.1 Q$	$860.6 + 247.7 Q$
AB	$163612 Q^{0.62}$	0	$738.2 + 39.9 Q$	Q
A1	$139753 + 17341.2 Q$	$0.23 + 0.088 Q$	$1852.8 Q^{0.42}$	$0.000885 + 0.000023 Q$
A2	$307785 + 33538.6 Q$	$0.16 + 0.18 Q$	$4259.3 Q^{0.41}$	$1158.4 Q^{0.62}$
A3	$198801 + 19934.9 Q$	$0.68 + 0.11 Q$	$3260.8 + 161.1 Q$	$2956.2 Q^{0.66}$
A4	$241226 + 33921.4 Q$	$0.26 + 0.16 Q$	$2783.4 Q^{0.47}$	$1694.4 Q^{0.65}$
A5	$269563 + 33561.5 Q$	$0.26 + 0.16 Q$	$2805.5 Q^{0.43}$	Q
B1	$232882 + 84335 Q$	$1.20 Q^{0.81}$	$2558.4 Q^{0.51}$	$0.0000662 + 0.00000036 Q$
B2	$241083 + 63200.5 Q$	$0.79 Q^{0.84}$	$2500.8 Q^{0.48}$	$2982.5 + 14255.3 Q$
B3	$241083 + 63200.5 Q$	$0.79 Q^{0.84}$	$2500.8 Q^{0.48}$	$4097.3 + 902.0 Q$
C1	$359744 + 84786.7 Q$	$0.76 Q^{0.80}$	$4574.8 Q^{0.45}$	$3525.8 + 895.8 Q$
C2	$349156 + 67047.4 Q$	$0.46 + 0.32 Q$	$6228.4 + 303.5 Q$	$3525.8 + 895.8 Q$
C3	$349156 + 67047.4 Q$	$0.46 + 0.32 Q$	$6228.4 + 303.5 Q$	$10499.7 Q^{0.73}$
C4	$395978 + 89419.7 Q$	$0.78 Q^{0.81}$	$4834.7 Q^{0.47}$	$10233.9 Q^{0.73}$
C5	$411240 + 89839.2 Q$	$0.78 Q^{0.81}$	$5093.2 Q^{0.47}$	$184641.1 + 15301.8 Q$
C6	$348852 + 53752.3 Q$	$0.50 Q^{0.84}$	$4292.9 Q^{0.44}$	$18720.2 + 14714.7 Q$
C7	$395350 + 59528.6 Q$	$0.78 Q^{0.81}$	$6959.8 + 360.6 Q$	$10336.4 Q^{0.73}$
C8	$413737 + 59627.4 Q$	$0.50 Q^{0.84}$	$4898.9 Q^{0.45}$	$18466.2 + 15301.5 Q$
D	$231495.0 Q^{0.66}$	$0.024 + 0.028 Q$	Q	$20047.7 + 14697.0 Q$
E	$629840 + 90719.4 Q$	$0.024 + 0.028 Q$	$0.00068 + 0.00058 Q$	$16491.9 Q^{0.68}$
F1	$327175 + 33438.9 Q$	$0.16 + 0.18 Q$	$1600.0 Q$	Q
F2	$327175.0 + 33438.9 Q$	$0.16 + 0.18 Q$	$2981.1 Q^{0.46}$	$0.00011245 + 0.00000014 Q$
G1	$210055 + 59204.6 Q$	$0.50 Q^{0.84}$	$3503.5 + 192.4 Q$	$2027.6 Q^{0.65}$
G2	$203714 + 56924.2 Q$	$0.50 Q^{0.84}$	$3360.1 + 183.9 Q$	$2027.6 Q^{0.65}$
G3	$209599 + 65633.4 Q$	$0.44 + 0.24 Q$	$3820.6 + 226.0 Q$	$8756.5 Q^{0.75}$
G4	$210055 + 59204.6 Q$	$0.50 Q^{0.84}$	$3503.5 + 192.4 Q$	$8756.5 Q^{0.75}$
H	$155767 + 37290.7 Q$	$0.49 + 0.16 Q$	$2031.1 Q^{0.42}$	$8756.5 Q^{0.75}$
I	$163270 Q^{0.88}$	$-0.17 + 0.021 Q$	$3746.2 Q^{0.72}$	$-3559.4 + 8110.1 Q$
				$15161.5 Q^{0.86}$

Table B-1 (Continued)

Unit Process	Base Capital Cost (BCC)	Land Requirement (LR)	Base Manhours (BMH)	Base Materials Cost (BMC)
J	$136587 Q^{0.52}$	$-0.081 + 0.047 Q$	$3043.2 Q^{0.41}$	$2399.3 + 39947.7 Q$
K	$93029.1 Q^{0.89}$	$-0.016 + 0.04 Q$	$3385.6 + 660.2 Q$	$2103.0 + 3490.0 Q$
R	$62270.5 + 5127.1 Q$	$0.21 + 0.018 Q$	$462.6 Q^{0.60}$	$-1748.7 + 2739.3 Q$
L1	$111168 + 23450.4 Q$	$0.40 + 0.09 Q$	$923.39 + 108.1 Q$	$1384.2 + 152.4 Q$
L2	$103721 + 21188.3 Q$	$0.32 + 0.09 Q$	$896.1 + 94.2 Q$	$1441.8 + 140.7 Q$
M1	$92686.9 + 12701.3 Q$	0	$2246.7 + 313.9 Q$	$2810.5 + 1025.8 Q$
M2	$101672 Q^{0.55}$	0	$2039.8 + 314.7 Q$	$2148.8 + 1025.0 Q$
N1	$-14885.6 + 57978.60 Q$	$1.5 + 1.9 Q$	$0.000518 + 0.00001161 Q$	$5.4 + 822.7 Q$
N2	$0.00000971 + 0.000000101 Q$	$0.287 + 0.0000972 Q$	$0.000490 + 0.000002 Q$	$0.001156 + 0.0000136 Q$
O1	$153201 + 27538.5 Q$	$-0.026 + 0.021 Q$	$2264.1 + 488.9 Q$	$6231.8 Q^{0.86}$
O2	$123189 Q^{0.71}$	$-0.026 + 0.016 Q$	$2510.8 + 514.9 Q$	$8059.1 + 3513.6 Q$
O3	$194601.0 + 45218.2 Q$	$-0.046 + 0.021 Q$	$2624.6 Q^{0.78}$	$9681.0 Q^{0.70}$
O4	$173784 + 37399.2 Q$	$-0.043 + 0.018 Q$	$2391.8 + 920.04 Q$	$6875.3 Q^{0.73}$
O5	$140189 + 22599.2 Q$	$-0.04 + 0.018 Q$	$1899.5 + 385.5 Q$	$6061.6 Q^{0.96}$
O6	$152608 + 22520.1 Q$	$-0.026 + 0.017 Q$	$2339.4 + 405.2 Q$	$5978.1 Q^{0.88}$
O7	$168827 + 31955.5 Q$	$-0.043 + 0.016 Q$	$3352.5 + 880.9 Q$	$7524.1 Q^{0.71}$
O8	$127953 + 13386.5 Q$	$-0.023 + 0.015 Q$	$1916.3 + 482.0 Q$	$4058.8 Q^{0.73}$
O9	$123931 + 18524.6 Q$	$-0.026 + 0.016 Q$	$1966.1 + 507.7 Q$	$4481.9 Q^{0.71}$
P1	$378837 Q^{0.54}$	$-0.084 + 0.015 Q$	$1280.6 + 509.2 Q$	$7136.4 + 728.8 Q$
P2	$386161 Q^{0.57}$	$-0.084 + 0.012 Q$	$1521.8 + 504.7 Q$	$9938.7 + 933.7 Q$
P3	$631877 + 67043.6 Q$	$-0.084 + 0.011 Q$	$1281.8 + 735.4 Q$	$20391.1 + 1974.8 Q$
P4	$410798 Q^{0.58}$	0	$1286.8 + 609.1 Q$	$13163.9 + 1214.7 Q$
P5	$286299 Q^{0.53}$	$-0.084 + 0.013 Q$	$1519.3 + 291.4 Q$	$7926.8 + 549.8 Q$
P6	$390808 Q^{0.54}$	$-0.084 + 0.015 Q$	$1433.9 + 492.1 Q$	$12703.3 + 1267.2 Q$
P7	$634606 + 65598.6 Q$	$-0.084 + 0.012 Q$	$1464.1 + 697.8 Q$	$8507.3 + 891.9 Q$
Q1	$422409 Q^{0.58}$	$-0.17 + 0.021 Q$	$3048.8 + 550.9 Q$	$4264.7 + 14698.8 Q$
Q2	$331428 Q^{0.54}$	$-0.17 + 0.021 Q$	$1770.4 + 356.9 Q$	$3267.5 + 7913.7 Q$
Q3	$534671 Q^{0.59}$	$-0.17 + 0.021 Q$	$1904.0 + 911.1 Q$	$-262.8 + 12458.3 Q$

APPENDIX B

COMPUTER PROGRAM FOR LAKE SIZE CALCULATIONS

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00010 C
00020 C ***** PROGRAM 115C *****
00030 C LAKE SIZE
00040 C OCTOBER, 1979
00050 C
00060 C CALCULATES VOLUME, DEPTH, AND SURFACE AREA OF A LAKE WITH SIDES AT
00070 C 3:1 SLOPE AND LENGTH EQUAL TO TWICE ITS WIDTH. MONTH 12 IS THE MONTH
00071 C OF MINIMUM LAKE DEPTH, MONTH 1 IS THE FOLLOWING MONTH.
00080 C
00090 DIMENSION EFFL(12),EVAP(12),AREA(12),VOLUME(12),DEPTH(12),
00100 + MONTH(12)
00110 REAL LENGTH
00120 DATA MONTH/'JAN','FEB','MAR','APR','MAY','JUN','JUL','AUG','SEP',
00130 + 'OCT','NOV','DEC',/,NO/'N'/
00140 KEYB=10
00150 IFRT=11
00160 NPG=1
00161 C
00170 10 WRITE(KEYB,12)
00180 12 FORMAT('NUMBER OF MONTH AFTER MINIMUM DEPTH MONTH. 1-12')
00190 READ(KEYB,*)MONTH1
00200 M=MONTH1
00210 DO 20 K=1,12
00220 WRITE(KEYB,15)MONTH(M)
00230 15 FORMAT('LOSS, IN. AND EFFLUENT, CU.FT., FOR 1-A4')
00240 READ(KEYB,*)EVAP(K),EFFL(K)
00241 M=M+1
00242 IF(M.EQ.13)M=1
00250 20 CONTINUE
00260 WRITE(KEYB,30)
00270 30 FORMAT('ESTIMATE OF MINIMUM LAKE DEPTH AND BOTTOM WIDTH, FT')
00280 READ(KEYB,*)DMIN,WMIN
00290 WIDTH=WMIN
00300 C
00310 C ITERATE UNTIL CALCULATED VOLUME OVER A YEAR EQUALS THE INITIAL VOLUME
00320 40 VOL1=DMIN*(2.*WIDTH*WIDTH+9.*WIDTH*DMIN+18.*DMIN*DMIN)
00330 AREA1=2.*WIDTH*WIDTH+18.*WIDTH*DMIN+36.*DMIN*DMIN
00340 VMIN=VOL1
00350 C
00360 DO 50 K=1,12
00370 VOL2=VOL1+EFFL(K)
00380 DEP2=(-2.*WIDTH*WIDTH+SQRT(4.*WIDTH**4+36.*WIDTH*VOL2))/(18.*WIDTH)
00390 AREA2=2.*WIDTH*WIDTH+18.*WIDTH*DEP2+36.*DEP2*DEP2
00400 AREA3=(AREA1+AREA2)/2.
00410 VOL3=AREA3*EVAP(K)/12.
00420 VOL1=VOL2-VOL3
00430 DEP1=(-2.*WIDTH*WIDTH+SQRT(4.*WIDTH**4+36.*WIDTH*VOL1))/(18.*WIDTH)
00440 DEPTH(K)=DEP1
00450 VOLUME(K)=VOL1/43560.
00460 AREA(K)=AREA3/43560.
00470 50 CONTINUE
00480 IF(VOL1.GE.VMIN)GO TO 70
00490 WIDTH=WIDTH-0.1
00500 GO TO 40
00501 C
00510 70 LENGTH=2.*WIDTH
00511 DO 80 K=1,12
00512 80 EFFL(K)=EFFL(K)/43560.
00520 C
00530 C OUTPUT
00540 WRITE(IFRT,100)NPG
00550 100 FORMAT(11//9X,'LOSS EFFLUENT SURFACE VOLUME DEPTH')

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00560      +10X,'IN      AC-FT  AREA,AC  AC-FT  FT')
00570      N=1
00580      DO 150 K=1,12
00581      IF(K.LT.MONTH1)M=13+K-MONTH1
00582      IF(K.GE.MONTH1)M=1+K-MONTH1
00590      WRITE(IPRT,110)MONTH(K),EVAP(M),EFFL(M),AREA(M),VOLUME(M),DEPTH(M)
00600 110  FORMAT(3X,A4,F6.1,F8.1,F10.1,F9.1,F7.1)
00610      IF(K.NE.3*N)GO TO 150
00620      WRITE(IPRT,120)
00630 120  FORMAT(' ')
00640      N=N+1
00650 150  CONTINUE
00660      WRITE(IPRT,160)DMIN,WIDTH,LENGTH
00670 160  FORMAT(//8X,'MINIMUM DEPTH,FT',F19.1/
00690      +8X,'CALCULATED BOTTOM WIDTH,FT',F9.1/
00700      +8X,'CALCULATED BOTTOM LENGTH,FT',F8.1)
00710      NPG=NPG+1
00720      IF(NPG.EQ.2)NPG=0
00730 C
00740      WRITE(KEYB,170)
00750 170  FORMAT('0ANOTHER CALCULATION ? Y OR N')
00760      READ(KEY3,180)IANS
00770 180  FORMAT(A1)
00780      IF(IANS.NE.NO)GO TO 10
00790      STOP
00800      END

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